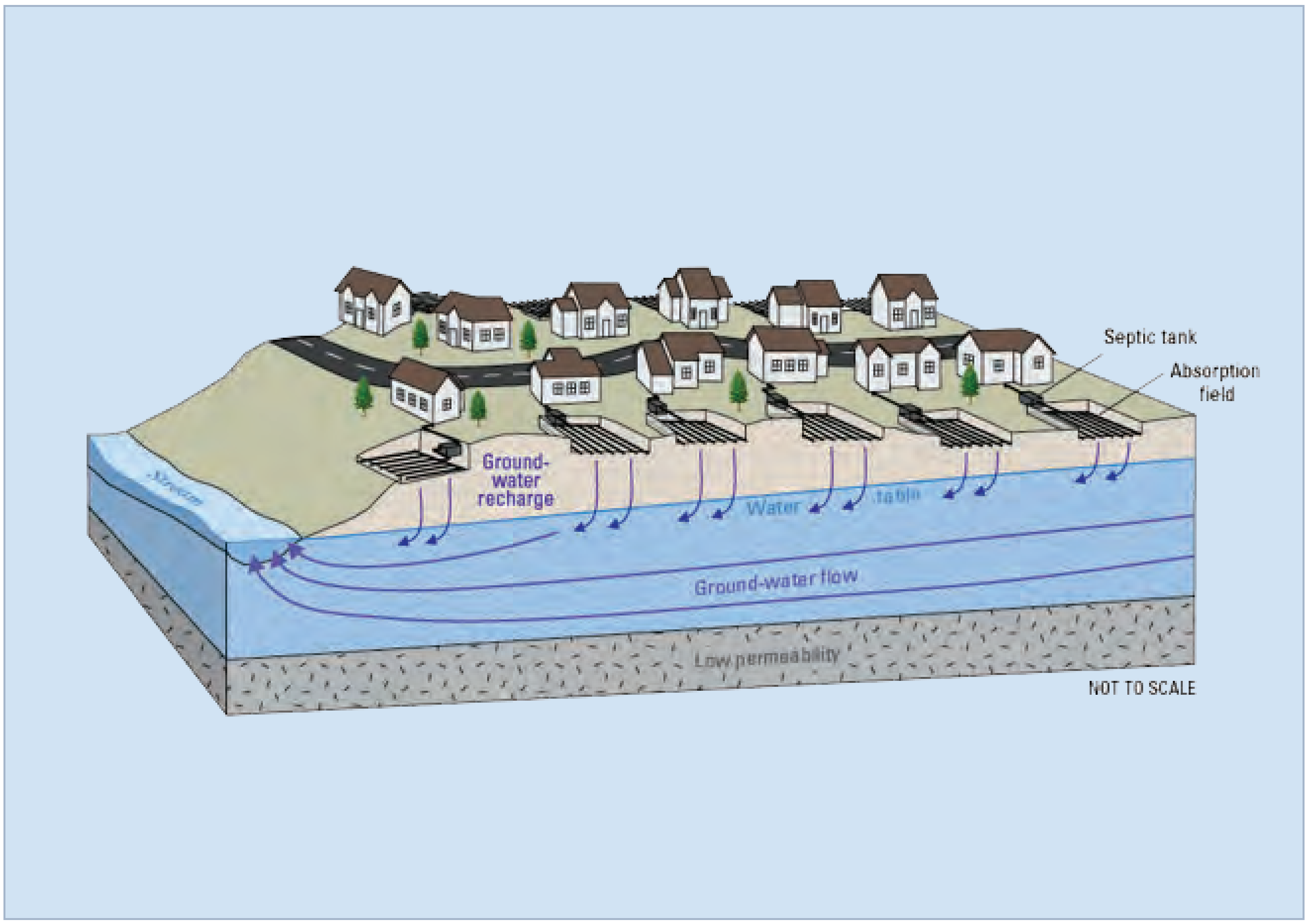
Conference Proceedings

Innovation in Soil-Based Onsite Wastewater Treatment



April 7-8, 2014 | Albuquerque, NM

Soil Science Society of America

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Program organizers: David Lindbo, John Buchanan,
Nancy Deal, and George Loomis

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Soil-Based Onsite Wastewater Treatment and the Challenges of Climate Change

Jose Amador[1*], George Loomis[2], and David Kalen[2]

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ABSTRACT

A quarter of the U.S. population relies on onsite wastewater treatment systems (OWTS) to provide soil-based dispersal and treatment of domestic wastewater. The current state of knowledge indicates that the presence of oxygen and optimal soil moisture conditions will enhance treatment mechanisms in the vadose zones beneath OWTS soil treatment areas. Regulatory codes are predicated on this basic understanding, and many OWTS are already installed under this long-standing paradigm. Climate change is real and here to stay – predicted warmer, and wetter or drier, climatic conditions will pose challenges to system treatment performance and longevity. The potential impact of climate change on OWTS may include elevated sea level and water tables, compromised separation distances, wetter/saturated soil pore space, lower O_2 solubility, higher soil microbial O_2 consumption due to higher soil temperatures, further reduction in levels of O_2 available for wastewater treatment – all of which can contribute to diminishing the infiltrative and water quality functions of OWTS. We need to recognize climate change as a real and imminent challenge, begin to understand these impacts more fully, and develop mitigation and adaptation measures that are sustainable and protective of public and environmental health.

Background

Nearly 25% of households in the United States rely on onsite wastewater treatment systems (OWTS) as their only means to treat wastewater (USEPA, 2011). The long-term sustainability of rural and suburban communities, and natural ecosystem health, is predicated on availability of clean ground and surface water. OWTS rely on soil-based wastewater treatment mechanisms to remove/inactivate pathogenic organisms, pharmaceuticals and personal care products; retain phosphorus; transform nitrogen species; and – degrade and assimilate organic material in wastewater. As an industry, we recognize the importance of soil physical, chemical and biological characteristics, maintaining proper separation to the water table and other restrictive layers, and system position on the landscape in achieving proper treatment and dispersal of wastewater. If soil and site conditions are adequate, we have a reasonable expectation of achieving treatment levels that are protective of watershed, public and environmental health.

Global Climate Change

Climate change is already with us, and has been for a while. One of its manifestations is significant variability in climatic conditions that differ regionally. In 2013 and early 2014, it is not difficult to point to atypical climatic conditions in many areas of the United States. Wetter and colder conditions have produced record-setting winter ice storms and snowfalls in nearly all areas of the U.S. except the southwest; sustained drought in the west; floods in large areas of the northwest; and, a high frequency of tornadoes in the so-called “tornado alley”.

These extreme climatic conditions unfortunately result in loss of property and lives. Intuitively, they also serve as additional stressors on OWTS soil treatment dynamics and, if these conditions persist over the long-term, threaten to reshuffle the soil-based wastewater treatment paradigm that all past, current and future system designs, regulations, and policies are based on. There is growing concern that the performance of existing and future OWTS will be compromised by changes in climate, leading to the degradation of the Nation's water resources and potential public health risks.

Climate Change Facts

Climate change models predict dryer and warmer conditions for some regions, as well as increased precipitation, sea level rise and/or higher temperatures, depending on current climate and geographic location. In dryer climatic regions, the predictions may lead to long-term drought conditions and warmer temperatures. For coastal areas, sea level rise models predict a 20 to 90 cm increase in sea level in mid and upper-Atlantic regions of the United Sates in the next century (Wu et al., 2009). Analysis of long-term trends in extreme precipitation events suggests that their frequency has increased in the continental U.S. and Canada (Kunkel, 1999).

The Intergovernmental Panel on Climate Change (IPCC), a scientific body jointly established in 1988 by the World Meteorological Organization (WMO) and the United Nations Environment Program (UNEP), actively studies climate change and provides policymakers with the most authoritative and objective scientific and technical assessments available. They indicate that climate change has impacted, and, will continue to impact, temporal and spatial patterns of precipitation and heat, as well as sea level rise (IPCC, 2007; 2013). In coastal areas of the U.S. this will likely translate into higher soil moisture and temperature, and higher water tables, all of which can have a negative effect on the functioning of OWTS and the quality of receiving waters.

In Rhode Island, long-term sea level data indicate a 25.6cm (9.9 inch) / 100 year rise in mean sea level over the period 1931 to 2009; this rate increased to 36.2cm (14.2 inch)/ 100 years for the period 1990 - 2009 (Boothroyd, 2012). Climate scientists have predicted that the rate of sea level rise will accelerate in the future (Glass and Pilkey, 2013). This will have regional implications, as a large portion of the glaciated New England region has fairly shallow water tables, and the anticipated rise in groundwater table will shorten the vertical separation distance between an OWTS soil treatment area (STA; aka drainfield) and the water table*. By 2100, the U.S. eastern seaboard, which extends from the Carolinas to New England, will experience a sea level rise 20 to 29 cm above the expected global increase, which most oceanographers predict to be about 1 meter (Sallenger et al., 2012). This would translate to an estimated 1.2-meter increase in sea level rise in hot spot areas. This will in turn cause a corresponding rise in near shore ground water tables, as the denser saltwater wedges landward under the less dense freshwater ground water lens.

* Separation distances range from 30 to 120 cm and vary by regulatory jurisdiction. For instance, in coastal critical resource areas of Rhode Island, regulations require 3 ft. (90 cm) separation distance for older OWTS systems and 4 ft (120 cm) for systems installed after 1989.

Expected Implications for OWTS Function

Wetter and Warmer

Wetter soil conditions due to increases in precipitation and/or rising water tables will likely leave less aerobic soil in the vadose zone for wastewater treatment processes to occur. This will be compounded by lower O_2 solubility and higher soil microbial O_2 consumption due to higher soil temperatures, further reducing levels of O_2 available for wastewater treatment. Wetter soil conditions and less available oxygen in soils beneath STAs will likely reduce nitrification efficiency (oxidation of ammonium to nitrate; an important goal of onsite wastewater treatment), potentially resulting in more ammonium entering ground waters and adjacent receiving water bodies, where elevated ammonium levels could cause toxicity to invertebrates and fishes. The lower levels of nitrate in ground water may result in a reduced potential for landscape-level denitrification of nitrate, placing additional stress on marine and brackish receiving water ecosystems.

Wet soil conditions will also lead to reduction in iron on soil particle surfaces, which may cause phosphorus attached to these soil surfaces to be solubilized and released into the soil pore water solution, becoming more mobile and leaching to ground water. As phosphorus moves with ground water to fresh surface water bodies, phosphorus enrichment will increase the potential for eutrophication, lowering water quality. Should those impaired receiving waters be drinking water reservoirs, water quality impairments will translate into higher water treatment costs.

Mechanical filtration of pathogenic bacteria and protozoan cysts by soil, and sorption of viruses to soil particles – the main mechanisms of pathogen removal and deactivation – are controlled by soils moisture levels, the vertical separation distance in STAs, and ultimately by the distance between the STA and receptor wells or receiving surface waters. Wetter soils conditions, a reduction in the vertical separation distance, and less available O_2 – resulting from higher water tables due to increased precipitation and/or sea level rise – are expected to compromise the effectiveness of these removal mechanisms. Under this scenario, similar negative impacts on treatment are likely to occur for pharmaceutical and personal care products (PPCP).

Furthermore, increases in temperature will interact with rising water tables to diminish the volume of aerobic soil in the vadose zone, both by lowering O_2 solubility in water and by decreasing the size of the vadose zone. Although the relative importance of wetter soils, higher water tables, sea level rise and increased temperature effects is likely to be different depending on the region, the overall pathogen removal functions of OWTS are expected to be impacted negatively by climate change. Over the long-term, the effects of climate change in humid regions receiving more precipitation and warmer temperatures are expected to result in complete loss of the infiltrative and water quality functions of OWTS.

Dryer and Warmer

Some regions that are already experiencing drier than normal conditions are expected to remain so, and to become warmer. All the expected issues discussed previously related to increased temperature – lower O_2 solubility, higher soil microbial O_2 consumption due to higher soil temperatures, further reduction in levels of O_2 available for wastewater treatment, and reduction in treatment potential – will also likely occur under this dryer and warmer climate change scenario. Under this situation, oxygen levels will likely be insufficient to efficiently nitrify ammonium, inactivate pathogens, and degrade PPCP. To compound this issue is the expectation that drier conditions will promote more water conservation measures in water-poor regions, which will likely produce lower carriage water volumes in households and subsequently higher wastewater constituent concentrations.

Meeting the Climate Change Challenges

In an effort to address the impacts of climate change on OWTS, a new USDA Hatch Multistate Project (NE-1045 Project) was developed in 2010 to gather the expertise of researchers and educators at Land Grant and Sea Grant institutions, as well as other higher educational institutes, to begin to address these issues. This five-year project entitled *Design, Assessment, and Management of Onsite Wastewater Treatment Systems: Addressing the Challenges of Climate Change* provides the opportunity for scientist engaged in OWTS research and outreach activities to work within their respective institutions and collectively (hence, the multistate aspect of the project) to addresses many of the unknowns about OWTS function relative to changing climatic conditions. Nineteen scientists representing these Land Grant /Sea Grant institutions – Cornell Univ., MSU, NCSU, OKSU, Rutgers Univ., UAZ, UGA, UK, UMN, UMO, URI, UTK – are participants in NE-1045; and, others are welcome to also join. Participation in this project comes with no assurances for institutional funding, but it has enabled the start of several important research efforts (some of this research presented at this conference) and has begun the task of informing the industry and practitioners of this important issue.

When Opportunity Knocks

The issue of climate variability and change confronts the OWTS industry and professional practitioners with a whole new set of challenges and opportunities, which if embraced, will position our industry to compete and pace well with other entities and disciplines, many of which have fully incorporated climate change planning and management in their future endeavors. If we choose to ignore the need, or deny the existence of climate change and the real and imminent issues it poses, we will lose hard-earned ground and credibility as an industry. The opportunity this presents to the decentralized wastewater industry is significant, as advance wastewater treatment will become even more important, in more watersheds and over larger land areas, as stressors to effective soil-based wastewater treatment begin to multiply. The following suggestions are offered as a means for our industry to proactively mitigate and adapt to the inevitability of climate changes:

- Ground truth your current understanding: read the science and become aware of the facts on climate. Climate change is real, and it’s here to stay.
- Educate yourself so you can inform your clients.
- Position yourself, your company, and/or your research to proactively address this issue.
- Avoid the “minimalist” mentality - increase separation and setback distances if and when you can.

- Map locations of the systems at risk.
- Account for projected sea level and ground water rise in your future designs.
- Base your decisions on risk management as a justification for making a more conservative and robust design.
- Improve on component-based treatment efficiency.
- Develop new technologies and approaches that are climate change-ready.
- Think about adding air to soil to counteract losses from increased temperatures and/or higher soil moisture.
- Utilize shallow soil dispersal of wastewater, which automatically increases separation distance and utilizes more biochemically reactive soil.
- Proactively manage soil moisture by timed-dosing.
- Research soil amendments that augment wastewater treatment potential.
- Support and focus research that addresses climate change issues.

ACKNOWLEDGEMENTS

The authors thank the Rhode Island Agricultural Experiment Station and Rhode Island Sea Grant Program for funding this effort. This document is produced under the auspices of the Rhode Island Agricultural Experiment Station, USDA Hatch Multi-State Project NE-1045.

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Community Septic System Owners Guide.

Sara Heger, University of Minnesota

ABSTRACT

This USDA grant funded project, led by the University of Minnesota (UMN), is developing a wastewater decision-making tool for consumers to help to transform rural wastewater management by developing a customizable Community System Owner's Guide (CSOG). At the time of preparing this paper, the project is at the end of year one of a three year project.

The primary deliverable of this project is a web-interface that allows an individual to produce an expert-driven and locally-customized manual (electronic or hard-copy) CSOG for any cluster soil-based wastewater treatment system in America. This tool will provide users with fundamental information about the operation and management of various wastewater management systems. A consultant, engineer, septic professional, facilitator, or even an educated community member will be able to use this tool to develop a management plan for either a new or existing community onsite wastewater treatment systems OWTS. The developer of any given CSOG will be able to assemble a professionally designed guide by selecting situation-specific boilerplate language and graphics and inserting customized content to integrate system-specific permit and ordinance requirements. Key partnerships in Arizona, Iowa, Michigan, Minnesota, and North Carolina, along with the US EPA, will be utilized to assure this grant will deliver a nationally relevant and locally customizable interface tool to facilitate the development of Community System Owner's Guides.

Engineering Design of a Modern Soil Treatment Unit

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ABSTRACT

The vast majority of existing onsite and decentralized wastewater treatment systems involve discharge of a partially treated wastewater into a subsurface trench or bed for infiltration and percolation to groundwater. Consistent with years past, most new systems are similarly configured. However, unlike years past where the primary goal was often just to achieve disposal, a modern goal is to design a soil treatment unit (STU) that can achieve tertiary treatment with natural disinfection. To broadly realize this goal and for STU systems to realize their full potential, the design process needs to become more rational and more uniform across practitioners and regulatory jurisdictions. In contrast to an empirical design process based largely on local experiences embodied in guidance prescribed in regulatory codes that have persisted for decades, rational design should be built on clear and compelling science and engineering underpinnings. Major research findings during the past decade or more have provided new insight into the key processes that govern performance of a STU. This knowledge base has enabled development of a more rational approach to engineering design of a modern STU. This paper provides a summary of the science underpinning effluent infiltration and purification in a STU. It also describes a rational approach for the engineering design of a modern STU as a unit operation within an onsite wastewater treatment system. The paper also highlights how such an engineering design approach has been adopted into regulations governing onsite system design and implementation.

INTRODUCTION

During the 20th Century, onsite wastewater systems serving homes and businesses in rural areas were almost always established so that wastes or wastewaters would be discharged below ground into subsurface soils. Early on, pit privies were built to receive human wastes and cesspools were developed to receive liquid wastes. Septic tanks were introduced to receive and treat liquid wastes before discharge. To enable discharge of septic tank effluent into subsurface soils, onsite systems were outfitted with what were referred to as seepage pits, leachfields, or drainfields. These systems were explicitly designed to be simple and cheap but also effective in keeping wastes away from people. However, they were not explicitly designed or implemented to achieve long-term treatment goals. During the latter decades of the 20th Century, increased water use and wastewater generation and more widespread use of disposal-based systems in a growing suburban America, led to problems – hydraulic malfunctions, groundwater contamination, and surface water quality deterioration.

In the modern world (i.e., the 21st Century), several hundred thousand new and refurbished onsite wastewater systems are established each year to serve homes and businesses in rural and suburban areas across the United States. A growing number of decentralized wastewater systems are being implemented to serve clusters of homes and businesses as well as developments and small towns. The vast majority of these new systems include a unit operation involving soil. In contrast to years past however, in the modern world the overall goal can be to achieve tertiary treatment with natural disinfection. Similar to a tank-based treatment unit, an unconfined soil profile can be conceptualized as a wastewater treatment unit operation that is designed to: 1) hydraulically process and purify the effluent within the soil profile to the extent needed to protect public health and water quality, 2) provide a long service life with low operation and maintenance requirements, 3) enable groundwater recharge, and 4) have an affordable cost

Source: Siegrist RL. 2014. Engineering design of a modern soil treatment unit. In: Innovations in Soil-based Onsite Wastewater Treatment, Proc. Soil Society Society of America Conference, Albuquerque, NM, April 6-7, 2014. 14 pp. (29Apr14)

(Siegrist, 2006; 2007). Using the terminology, Soil Treatment Unit (STU), reflects this conceptualization.

To develop a fundamental understanding of the principles and processes important to the design and performance of soil treatment units used within onsite and decentralized wastewater systems, research has been ongoing for more than a decade within the Small Flows Program at the Colorado School of Mines in Golden, Colorado, USA (Siegrist et al., 2012; 2013; 2014). Recent and ongoing research has been focused on soil treatment of different quality effluents using two system approaches: 1) effluent dispersal into a soil profile using shallow trenches outfitted with infiltration chambers and 2) effluent dispersal into the rhizosphere using drip tubing with pressure-compensating emitters.

Within the Small Flows Program, STU research has included laboratory experiments, controlled field experiments with pilot-scale units, field monitoring of full-scale systems, and analytical and numerical modeling. The program of research has been conceived to develop a quantitative understanding of soil treatment unit design and performance including flow and transport and the removal of pollutants and pathogens as affected by soil properties, system features, effluent quality and loading, and other design factors and environmental conditions. The research has also developed models and decision-support tools for soil treatment unit applications. This paper provides highlights of the research carried out. Additional details on a given topic may be found in the literature cited. Due to space limitations, this paper is focused on soil treatment using subsurface infiltration trenches. While many of the principles and processes are also applicable to soil treatment using drip dispersal of effluent into the rhizosphere, this soil treatment approach is not explicitly covered in this paper.

KEY PROCESSES AND ACHIEVABLE PERFORMANCE

Key Processes. Wastewaters treated by onsite and decentralized systems can contain a variety of pollutants and pathogens at low to very high levels (Table 1). The nature of the source and the water-use and waste-generation characteristics determine the composition of the wastewater that must be handled by the system. Traditional constituents of concern include oxygen consuming compounds, particulate solids, nitrogen, phosphorus, heavy metals, bacteria and viruses (Lowe et al., 2009). Emerging constituents of concern include an array of organic compounds (e.g., caffeine, nonylphenols, Tricosan) that can be referred to as trace organics due to their relatively low concentrations. Trace organics associated with consumer product chemicals can routinely occur at varied levels depending on the source (e.g., residential dwellings vs. commercial establishments) (Conn et al., 2006; Conn et al., 2010a). Pharmaceuticals, pesticides and flame retardants can also occur, but much less pervasively and typically at much lower levels (Conn et al., 2010a).

During treatment of wastewater effluent in a STU a dynamic interaction of a complex set of hydraulic and purification processes at the soil infiltrative surface and in the soil profile govern system function and performance (Fig. 1). When a partially treated effluent (e.g., septic tank effluent (STE)) is applied to soil, effluent infiltration and percolation with eventual ground water recharge involves: 1) effluent infiltration into soil pore networks; 2) effluent water movement within a soil profile (percolation – movement within the pore network, groundwater recharge – transport into groundwater, evapotranspiration – transport up and out of the soil profile); and 3) effluent pollutant and pathogen removal reactions (kinetic reactions (e.g., biodegradation), capacity-based reactions (e.g., filtration, sorption), plant-based reactions (e.g., nutrient uptake)).

These processes can interact in a dynamic manner, evolving as the STU matures from startup through the first year(s) of operation. Process 1) involves biozone genesis which directly affects processes 2) and 3). Biozone genesis has been characterized to include three processes: a) biofilm formation, b) biomat development, and c) humic substance-like material development (Siegrist, 2007; McKinley and Siegrist, 2010). A decline in the infiltrability of the soil infiltrative surface (i.e., the native soil's capacity to infiltrate water if made freely available) is caused by soil clogging due to biomat formation and pore-filling at and near the location where effluent enters the soil pore network. The rate and extent of infiltration rate (IR) decline has been attributed to the hydraulic loading rate and effluent quality applied to the soil (as measured by total suspended solids (TSS) and total biochemical oxygen demand (BOD) (carbonaceous BOD plus nitrogenous BOD) (Siegrist and Boyle, 1987; Siegrist et al., 2001; Beach et al., 2005; Beal et al., 2005; Van Cuyk et al., 2005; Lowe and Siegrist, 2008). After a period of operation, a STU can experience a sufficient decline in infiltrability such that intermittent or continuous ponding of the infiltrative surface can ensue. This can result in a hydraulic head that helps enable infiltration of the daily hydraulic loading rate. The time to development or sustained occurrence of ponding does not necessarily correlate with long-term hydraulic or treatment performance. A STU can operate effectively in an intermittent or continuously ponded condition for an indefinite period of time. However, under some conditions, such as when higher strength wastewater or higher daily loading rates occur compared to design assumptions, or after an extended period of continuous use (e.g., 20 years or more), excessive soil clogging can occur. This can lead to hydraulic dysfunction where the infiltrative surface becomes so impermeable that that the daily wastewater loading can no longer be fully infiltrated.

Pollutants and pathogens can be removed in a STU by many physical-chemical and biological processes. BOD removal can occur by biodegradation in biofilms that grow on soil grains and within soil organic matter. Suspended solids can be removed by physical filtration and absorption followed by biodegradation. Reduced forms of nitrogen (e.g., NH_4^+) can be biologically oxidized completely and some total N can be removed by biodenitrification. Phosphorus removal varies widely depending on soil mineralogy and its P-sorption properties. Pathogens such as parasites and bacteria can be filtered out and die-off while virus can attach to grain surfaces and be inactivated. Purification of trace organic compounds (e.g., caffeine, nonylphenols, Tricosan) principally occurs by sorption and biodegradation Conn et al. 2010b). Biozone genesis (as described earlier) can provide more rapid and extensive treatment of a wastewater effluent by enhancing sorption, nitrification/denitrification, and biological decay at and near the soil infiltrative surface (Siegrist, 1987; Van Cuyk et al., 2001; Siegrist et al., 2005; Van Cuyk et al., 2005; Van Cuyk and Siegrist, 2007; Tomaras et al., 2009).

Soil treatment units are often expected to achieve tertiary treatment and natural disinfection. For this to occur, highly unsaturated flow under aerobic conditions is normally critical. This flow regime facilitates contact between wastewater constituents and the soil grain surfaces and their associated biofilms and provides for a relatively long period for treatment processes to occur (Bouma, 1975; Emerick et al., 1997; Schwager and Boller, 1997; Van Cuyk et al., 2001; Siegrist et al., 2001; Van Cuyk et al., 2004; Van Cuyk and Siegrist, 2007). Unsaturated flow conditions can be achieved by hydraulic design if the design hydraulic loading rate (HLR_D) is limited to a small fraction of the soil's saturated hydraulic conductivity (Ksat) (e.g., $HLR_D < 0.001Ksat$) and application is achieved by intermittent dosing through pressurized piping networks. Also, over time, effluent infiltration can lead to soil clogging and unsaturated flow conditions irrespective of hydraulic design attributes.

Achievable Performance. A soil treatment unit can be designed and implemented to reliably achieve tertiary treatment with natural disinfection over a service life of 20 years or more. Key conditions that are required to achieve this performance level include: 1) the hydraulic conductivity of the infiltrative surface zone is not dramatically reduced by compaction, smearing, or solids deposition during installation and startup; 2) the HLR_D and/or concentrations of pollutants that cause soil clogging are not excessive compared to design assumptions; 3) there is an adequate soil profile depth for treatment - depending on effluent loading rate and quality, a certain depth of unsaturated aerobic soil is needed for treatment to occur; 4) there is unsaturated flow in the soil profile with long travel times so kinetic processes can achieve pollutant removals (e.g., removal of BOD, NH_4^+, Fecal coliforms); 5) there is an adequate volume of soil profile to provide soil grain surface area for sorption processes (e.g., P removal); and 6) subsurface conditions are conducive to treatment (e.g., circumneutral pH, high Eh, moderate temperatures, no biotoxins). The treatment efficiencies normally expected of a well designed and properly operated soil treatment unit are given in Table 1.

The inherent nature of a STU can complicate the use of quantitative treatment expectations (e.g., Table 1) and the ability to verify their achievement through monitoring. For a STU the end-of-pipe equivalent is the soil solution at some depth (e.g., 0.6 m below the infiltrative surface which might be where shallow groundwater exists). Depending on the environmental setting, further purification can occur as reclaimed water moves through the deeper vadose zone and migrates through ground water (e.g., to a well or into surface water). This assimilation of effluent from a STU and attenuation of residual constituents of potential concern can be critically important to achieving public health and water quality protection goals (e.g., attenuation of nitrate-nitrogen, virus, trace organics) (Fig. 1). Use of a mass discharge approach for evaluating treatment effectiveness and impacts can incorporate and account for this attenuation within the vadose zone and groundwater system (Siegrist et al. 2012).

ENGINEERING DESIGN OF A MODERN STU

The rational design and implementation of a modern STU at a particular site requires consideration of several key elements: 1) treatment goals and method of assessment, 2) suitability of site conditions and soil properties, 3) treatment required prior to application to a soil infiltrative surface, 4) architecture of the soil infiltrative surface, 5) effluent application rates for infiltration area sizing, 6) depth and properties of soil required beneath the infiltrative surface, 7) geometry and landscape placement, 8) effluent application and distribution, 9) options to ensure long-term service, 10) installation, startup and operation, and 11) monitoring and performance assurance. This section provides a summary of several of these elements.

Assuming a reasonably accurate estimate of the design daily flow rate for a particular application has been made, selecting a hydraulic loading rate for design of a STU (HLR_D) can be one of the most difficult steps in the design process. Some general considerations regarding selection of an HLR_D are outlined in Table 2. In contrast to some attempts to assign numerous HLR_D to different soils based on small differences in physical properties such as soil texture and structure (USEPA, 2002; Siegrist, 2006), the process proposed herein includes a simplified approach where soil is classified in three major groups as shown in Table 3. This is based on research that has revealed that the capacity of a soil infiltrative surface to accept wastewater effluent during long-term operation (so-called, long-term acceptance rate or LTAR) is relatively insensitive to native soil properties for soils with saturated hydraulic conductivities ranging from ~5 to 2500 cm/d. It is noted that soil morphology may be sufficient to classify soil profiles for this purpose and that even a crude percolation test may be of some value for the coarse discrimination included in this classification scheme.

For a given soil class, a maximum HLR_D value (HLR_D-max) should be set based on the recognition that even very high quality effluents (e.g., sand filter effluent, membrane bioreactor effluent) can cause soil clogging and permeability loss if the HLR_D is too high compared to the clean-water hydraulic conductivity of the native soil (Van Cuyk et al., 2005). A HLR_D-max value needs to be set to sustain effluent infiltration at that rate during long-term, continuous application (i.e., routine daily operation) for a reasonable design life (e.g., 20 years) even if highly treated effluent is applied to the soil. HLR_D-max values for any effluent applied to an open soil infiltrative surface are set such that the HLR_D will not exceed 5 to 10% of the clean water hydraulic conductivity of the soil infiltrative surface zone prior to wastewater effluent application (Table 3). Effluent classification includes three major effluent types as presented in Table 4. This effluent classification is based on differences in the effluent composition with respect to key soil clogging parameters (cBOD, TKN, TSS) and oxygen consuming materials that can affect the aeration and biochemical status of the soil profile (cBOD, TKN) (Siegrist, 1987; Siegrist and Boyle, 1987; Van Cuyk et al., 2005). The base HLR_D's are then established for the three primary soil classes and three effluent types. These base HLR_D values are for an open horizontal infiltrative surface and set to limit the applied loadings of total BOD and TSS to rates that can normally be assimilated by an aerobic soil environment. To facilitate lower soil water contents and profile aeration status, these base HLR_D values are constrained so that regardless of effluent quality, they do not exceed the HLR_D-max - an upper limit set at 5 to 10% of the saturated hydraulic conductivity of the soil infiltrative surface zone prior to any effluent loading and assuming no construction damage.

Determining the area required for effluent infiltration can be done using Equation 1, where A_{IS} = area of the infiltrative surface (ft^2 or m^2), Q_D = design daily flow (gal/day or L/d), HLR_D = hydraulic loading rate for a soil class and effluent (gpd/ft^2 or L/m^2 d), and EF = infiltration efficiency factor (EF = f(construction, operation (-); varies from 1 to ~0).

$$A_{IS} = \left[\frac{Q_D}{(HLR_D)}\right]\left(\frac{1}{EF}\right) \tag{1}$$

The efficiency factor in Equation 1 is derived from design or operational features such as those outlined in Table 5 that can impact STU performance. It is noted that factors of safety could be applied at this stage in the design process (assuming they are not embedded in the estimate of design flow or otherwise elsewhere during design). Other key design elements and parameter values that need to be specified include those associated with the STU layout and installation attributes (Table 6) as well as the method of effluent application and distribution within the STU (Table 7).

Analytical and numerical models of varying scope and complexity are available to aid analysis and design of an isolated system or clusters of STU as well as for assessment of potential benefits and impacts at the local, development, and watershed scale (e.g., Beach and McCray, 2003; McCray et al., 2005; Poeter et al., 2005; Radcliffe et al., 2005; Siegrist et al., 2005; Pang et al., 2006; Bumgarner and McCray, 2007; Heatwole and McCray, 2007; Radcliffe and West, 2007; Beal et al., 2008; Finch et al., 2008; McCray et al., 2009; 2010; Geza et al., 2010; 2013). Modeling tools are also available to evaluate the environmental effects of onsite and decentralized systems vs. centralized facilities within a particular planning area (e.g., Kellogg et al., 1997; Siegrist et al., 2005; Lemonds and McCray, 2007; Geza et al., 2010).

All STUs require some level of operational control and process monitoring to help assure that the performance objectives embedded in the design process are actually achieved. However, the nature and extent of operational control and process monitoring is highly dependent on the performance objectives, STU design attributes, and the environmental and regulatory setting. While beyond the scope of this paper, it is emphasized that management, at a level appropriate to the complexity of the STU and overall system design combined with the sensitivity and risk associated with the environmental setting can be critical to ensuring proper design and implementation to achieve a desired performance.

ENGINEERING DESIGN AND REGULATORY REFORM

The translation of research findings and improved scientific understanding into design procedures such as outlined in the previous section and then into regulations governing onsite and decentralized system design and implementation is a difficult and time-consuming process. Research findings do not automatically yield advances. Clear and compelling findings can foster advances. But improved practices also require translation of findings so they convey knowledge and know-how to designers, contractors, regulators, and policy makers and enable adoption of findings into modern regulations and requirements. As an example, in Colorado regulatory reform has occurred but with considerable effort on the part of many over nearly a generation. This section highlights aspects of a major reform of Colorado regulations that included engineering design of a soil treatment unit (referred to in the Colorado regulations as soil treatment area). In the pre-modern era in Colorado, onsite regulations were promulgated at the state level and contained in “Guidelines on Individual Sewage Disposal Systems” (e.g., CDPHE 2004). Recognizing the need for re-evaluation of the individual sewage disposal system regulations, the Colorado Department of Public Health and Environment (CDPHE) formed a Steering Committee to explore the matter. The Steering Committee developed a set of recommendations concerning necessary and appropriate changes that needed to be made to the individual sewage disposal system (ISDS) regulations and published their report in 2002 (CDPHE, 2002). Over the subsequent decade, a huge and dedicated effort on the part of scores of designers, contractors, regulators, business owners, and professional groups (e.g., Colorado Professionals for Onsite Wastewater) led to a modern set of regulations being developed, broadly vetted and eventually adopted as Colorado Regulation 43 – On-Site Wastewater Treatment System Regulation (CDPHE, 2013).

Colorado Regulation 43 includes many elements that have been modernized based on improved understanding gained through scientific research and practical experiences. One element directly related to this paper has to do with the sizing of a soil treatment unit, which in the regulations is referred to as a soil treatment area. Regulation 43 includes five different treatment levels based on the effluent concentrations expected in terms of carbonaceous BOD_5, total suspended solids, and total nitrogen (Table 8). Higher treatment levels can reduce the required horizontal and vertical setback distances. The base soil treatment area size is determined based on the estimated design flow and a LTAR determined for each of five treatment levels and five major soil conditions (Table 9). As noted earlier in this paper, effluent quality interacts with hydraulic loading rate in determining the rate and extent of wastewater-induced soil clogging during effluent infiltration. The LTARs range from 0.10 to 1.40 gpd/ft^2 and the relative effect of a higher treatment level enabling a higher LTAR is greater for soil conditions characterized by higher clean water hydraulic conductivities (Table 9). Adjustments to the base soil treatment area size are made for the geometry of the area (e.g., trenches require less infiltrative surface than beds), the method of effluent application (e.g., pressure dosing requires less area than gravity delivery), and the architecture of the infiltrative surface (for TL 1 only) (e.g., chamber outfitted trenches require less area than rock-filled trenches) (Table 10).

All of the elements just described are related to soil treatment area sizing in Colorado Regulation 43. One might have differing opinions about, or even debate, the specific values associated with different parameters, but the framework of Colorado Regulation 43 is consistent with the science-based engineering design of a modern soil treatment unit as highlighted earlier in this paper.

SUMMARY AND CONCLUSIONS

Major research findings during the past decade or more have provided new insight that enables engineering of a modern STU to achieve a needed performance. This paper presents an engineering approach and provides guidance values for design of a STU involving subsurface infiltration and percolation for ground water recharge to achieve long-term hydraulic performance (e.g., 20 years or more) while also providing tertiary treatment with natural disinfection within the STU and during assimilation into the subsurface environment. It is recognized that there are many unique situations and varied inter-related issues that can impact engineering design approaches and criteria for STU (e.g., the approach and conservatism in estimating design daily flows, confidence is assuring performance of tank-based treatment units, and so forth). However, this does not negate the need for and ability to develop a more rational generalized approach and set of guidance values. Proposed elements and values for a rational design approach have been formulated and evolved over the past five years or more by this author and others. Regulatory reform has begun and Colorado Regulation 43 is an example of how regulations can adopt a framework for science-based engineering design of a modern soil treatment unit.

ACKNOWLEDGMENTS

The author acknowledges the collaboration and contributions of faculty colleagues at CSM including: John McCray, Kathryn Lowe, Mengistu Geza, Junko Munakata-Marr, and Tzahi Cath as well as Jörg Drewes, John Spear, and Eileen Poeter. Scores of students have completed the research for their M.S. or Ph.D. degrees at CSM and they are acknowledged for their efforts to help advance the science underlying soil-based treatment systems. Several government organizations have enabled the research to be accomplished including the environmental health departments in Summit County, Jefferson County, and Tri-County, Colorado. Finally, numerous sponsors have supported the research upon which this paper is based through in-kind donations or grants and contracts, including U.S. EPA, U.S. Geological Survey, Water Environment Research Foundation, Infiltrator Systems Inc., and SCG Enterprises Inc.

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Figure. 1. Illustration of the environmental compartments within a modern soil treatment unit during effluent water flow and transport with pollutant and pathogen removal reactions. (Note: Q_I = effluent flow rate to the STU, C_I^i = influent concentration, R = reaction in a compartment, k = reaction rate, θ = retention time, C = concentration leaving the compartment, where the subscript (e.g., 1) designates a compartment and the superscript (e.g., i) designates a constituent).

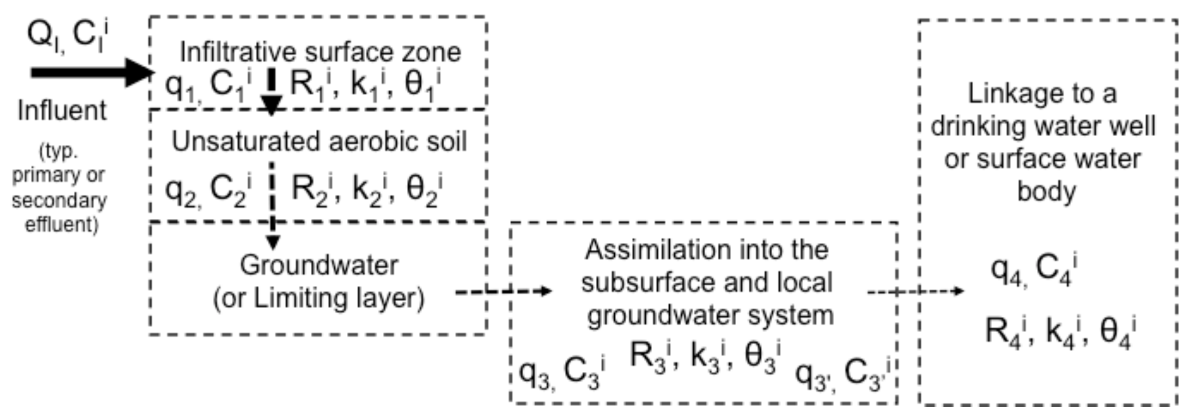

Table 1. Wastewater constituents and treatment expectations from a well-designed and properly operated soil treatment unit treating 1 to 5 cm/d of domestic septic tank effluent (Siegrist et al., 2012).

Constituents of concern	Basis for concern over wastewater constituent	Example unit of measure (units)	Domestic septic tank effluent [1]	Treatment efficiency in a STU [2]
Oxygen demanding substances	Can create anoxic or anaerobic conditions and can contribute to soil clogging	BOD_5 (mg/L)	140 to 200	>90%
Particulate solids	Contributes to soil pore filling and accelerated soil clogging	TSS (mg/L)	50 to 100	>90%
Nitrogen	Can contribute to oxygen demand, can be toxic via drinking water ingestion, can upset ecosystems	Total N (mg-N/L)	40 to 100	10 to 20%
Phosphorus	Can cause increased productivity in sensitive surface waters	Total P (mg-P/L)	5 to 15	100 to 0%[3]
Bacteria	Infectious disease transmission via drinking water, contact with seepage, or recreational waters	Fecal coliforms (org./100 mL)	10^6 to 10^8	>99.99%
Virus	Infectious disease transmission via drinking water, contact with seepage, or recreational waters	Specific virus (pfu/mL)	0 to 10^5 (episodically high levels)	>99.9%
Heavy metals	Potential toxicants to humans by ingestion in drinking water or to ecosystem biota	Individual metals (ug/L)	0 to low levels	>99%
Trace organic compounds	Potential health effects to humans by ingestion of drinking water or vapor inhalation during showering or effects to ecosystem biota	Organics in consumer products, pharmaceuticals, pesticides, flame retardants (ng/L or ug/L)	0 to trace levels	Low to >99%[4]

[1] Note: STE concentrations given are representative of those for residential dwelling units. However, commercial sources such as restaurants can produce STE that is markedly higher in some pollutants (e.g., BOD_5, COD, TSS, trace organics) while other sources can produce STE that is markedly lower in some pollutants (e.g., laundry can have lower total nitrogen and pathogen levels). [2] Efficiencies given are representative of concentrations in soil solution at 60 to 90 cm depth in a well-designed, installed and operated STU. [3] P-removal is highly dependent on media sorption capacity and P loading rates and time of operation. [4] Removal of trace organic compounds (e.g., nonylphenol, Triclosan, EDTA, caffeine) is highly dependent on the properties of the organic compound and conditions within the soil treatment unit (e.g., conditions conducive to sorption and biotransformation during adequately long hydraulic retention times).

Table 2. General considerations related to setting HLR_D for sizing a STU (after Siegrist, 2007).

Basis for selecting the HLR_D for sizing a STU
1. STU sizing should be based on horizontal infiltrative surface area since the effluent flow regime in the soil profile will be more uniformly unsaturated and more predictable; this sizing approach also reserves sidewall areas for handling peak flows
2. HLR_D is not just an inherent property of a soil, but is "system conditional" and depends on soil profile conditions, effluent type, operation, etc.
3. HLR_D is very dependent on effluent quality - Even for high quality effluents, the HLR_D must be a small fraction (e.g., $\leq$ 5 to 10%) of the saturated hydraulic conductivity of the infiltration zone.
4. The HLR_D must not exceed the hydraulic and treatment capacity of the soil profile and entire site: (a) Recognize potential low permeability zones and shallow groundwater so HLR_D does not cause excessive groundwater mounding. (b) For treatment, maintain HLR_D to provide adequate travel time, aeration, and soil contact volume.

Table 3. Simplified soil classification strategy and maximum HLR_D for effluent infiltration irrespective of effluent quality (after Siegrist, 2007).

Soil profile class	Representative soil textures	Representative clean water hydraulic conductivity (Ksat)	HLR_D - max (regardless of effluent quality)
Class I	Sand, loamy sand	1000 cm/d ($250\ gpd/ft^2$)	50 cm/d ($12.5\ gpd/ft^2$)
Class II	Sandy loam, loam, silt loam	100 cm/d ($25\ gpd/ft^2$)	10 cm/d ($2.5\ gpd/ft^2$)
Class III	Silty clay loam, clay loam	10 cm/d ($2.5\ gpd/ft^2$)	1 cm/d ($0.25\ gpd/ft^2$)

Table 4. Effluent classification and base daily HLR_D to account for key pollutants that control wastewater-induced soil clogging (after Siegrist, 2007).[1]

Effluent type	Effluent concentrations (mg/L)	Example unit operations for each effluent type	Class I soil	Class II soil	Class III soil
Type I	$cBOD_5$ =150 TKN =60 TSS =75	Septic tank (anaerobic bioreactor) with effluent screen	4 cm/d ($1.0\ gpd/ft^2$)	2 cm/d ($0.5\ gpd/ft^2$)	0.5 cm/d ($0.12\ gpd/ft^2$)
Type II	$cBOD_5$ =30 TKN =5 TSS =30	Aerobic treatment unit; Constructed wetland	10 cm/d ($2.0\ gpd/ft^2$)	4 cm/d ($1.0\ gpd/ft^2$)	0.5 cm/d ($0.12\ gpd/ft^2$)
Type III	$cBOD_5$ =5 TKN =5 TSS =5	Packed bed filter; Membrane bioreactor	20 cm/d ($4.0\ gpd/ft^2$)	4 cm/d ($1.0\ gpd/ft^2$)	1.0 cm/d ($0.25\ gpd/ft^2$)

[1] Note: HLR_D's are for determining the size of an open horizontal infiltrative surface based on year-round, normal usage, over a 20-year service life.

Table 5. Illustration of effluent infiltration efficiency factors used for infiltrative surface area sizing to account for STU construction and operation (after Siegrist, 2007).[1]

STU feature	Factor	Rationale
Construction impacts	0.1 or less	Account for the loss in clean-water Ksat due to compaction and smearing during installation.
Infiltrative surface architecture	0.50 - 0.75	Account for loss in LTAR due to solid objects including effects of fines and embedment and greater difficulty for monitoring and rehabilitation.
	1.0	Open infiltrative surface established with a chamber or similar technology
Discontinuous operation during normal 20-yr life	1.5 - 2.0	Account for elevated hydraulic and treatment capacity due to extended rest periods during cyclic operation; e.g., 1 year online and 3 years offline.
Relatively shorter design service life	2.0 - 4.0	Account for higher capacity even at higher HLR_D during only a short (1- to 5-year) design life.

[1] The efficiency factors shown in this table are for illustrative purposes only.

Table 6. Design guidance for STU layout and installation attributes (after Siegrist, 2007).

General design attribute	Design guidance
Landscape position	Place in well drained, upslope locations and orient infiltration units along contours to minimize linear-loading rates, particularly where shallow zones of lower permeability soils exist to minimize risk of mounding due to perching or water table elevation.
Geometry of the infiltration unit and placement in the soil profile	Infiltration trenches strongly preferred with trench width $\leq$90 cm and sidewall height $\leq$60 cm. Avoid large squarish beds, especially for Type I effluents. Shallow placement in the soil profile, ideally in the rhizosphere, but no deeper than 90 cm below ground surface, except to overcome low permeabilty layers or ground freezing concerns.
Minimum, separation to a limiting condition in the soil profile (e.g., ground water, bedrock)	Type I to III effluents in Class I soil = $\geq$60 cm Type I to III effluents in Class II and III soil = $\geq$90 cm
Desired service life from the STU	Design for long-term service (e.g., 20 years or more) should include plans for rejuvenation of treatment capacity and/or "reserve area" for installation of new infiltration units.

Table 7. Design guidance for effluent application to a STU (after Siegrist, 2007).

Effluent application	Design guidance
Delivery method to online components of the STU	Dosed application, such as provided by a pump: Class I soil = $\geq$ 4 doses per day and Class II and III soil = $\leq$ 2 to 4 doses per day.
Equalized application to all online components (e.g., trenches that are intended to be operational)	Type I effluent - soil clogging will enable more uniform infiltration via bottom infiltrative surfaces. Type II and III effluents - engineering (e.g., pressurized dosing) should attempt "uniform distribution" at startup and where clogging may be retarded or absent.
Effluent application rates during dosing via pressurized piping	To achieve uniform distribution to the infiltrative surface at startup, the instantaneous dosing rate should be $\geq$ the soil's Ksat for clean water (e.g., 1 gpm from an orifice to infiltrate 10 ft^2 provides an instantaneous dosing rate of 144 gpd/ft^2 or 0.007 cm/s (similar to a Class I soil Ksat)).
Uniform effluent distribution to online components	For sites with treatment limitations (e.g., limited unsaturated soil depth, shallow ground water with nearby drinking water wells), consider engineering-enhanced distribution; e.g., pressurized distribution networks with spray nozzles within a chamber-outfitted trench.
Cyclic loading of only part of the STU	Use dosing and sequential application of effluent to portions of the STU (e.g., 1 of 4 trenches) to achieve higher cumulative volume treated per unit area per time during soil clogging development. During long-term resting (resting cycle over $\geq$ 12 mon), soil infiltrative capacity can be restored.

Table 8. Different treatment levels based on concentrations of $cBOD_5$, TSS, and Total N (CDHPE, 2013).[1]

Treatment level	$cBOD_5$[1] (mg/L)	TSS (mg/L)	Total Nitrogen (mg/L)
TL 1 [2]	145	80	60 – 80
TL 2	25	30	60 – 80
TL 2N	25	30	>50% reduction [3]
TL 3	10	10	40 – 60
TL 3N	10	10	20

[1] Source: Table 6-3. Colorado Reg. 43. June 2013. [2] $cBOD_5$ can be estimated as 0.85 x total BOD_5. [3] Values for TL 1 are typical but design must account for site-specific information. [4] NSF/ANSI Standard 245 – Wastewater Treatment Systems – Nitrogen Reduction requires reduction of 50% rather than achieving a specific value.

Table 9. Hydraulic loading rates for design based on long-term acceptance rates for different soil conditions and treatment levels (CDHPE, 2013).[1]

Soil Type, Texture, Structure and Percolation Rate Range					Long-term Acceptance Rate (LTAR); Gallons per day per square foot				
Soil type	USDA soil texture	USDA soil structure-shape	USDA soil structure-grade	Percolation rate (MPI)	Treatment Level 1[2]	Treatment Level 2[2]	Treatment Level 2N[2]	Treatment Level 3[2]	Treatment Level 3N[2,3]
0	Soil Type 1 with more than 35% Rock (>2mm); Soil Types 2-5 with more than 50% rock (>2mm)	-	0	<5	Min. 3-ft. deep unlined sand filter required	Minimum 2-foot deep unlined sand filter required[2]			
1	Sand, Loamy Sand	-	0	5-15	0.80	1.25	1.25	1.40	1.40
2	Sandy Loam, Loam, Silt Loam	PR (prismatic) BK (blocky) GR (granular)	2 (Moderate) 3 (Strong)	16-25	0.60	0.90	0.90	1.00	1.00
2A	Sandy Loam, Loam, Silt Loam	PR, BK, GR 0 (none)	1 (Weak) Massive	26-40	0.50	0.70	0.70	0.80	0.80
3	Sandy Clay Loam, Clay Loam, Silty Clay Loam	PR, BK, GR	2, 3	41-60	0.35	0.50	0.50	0.60	0.60
3A	Sandy Clay Loam, Clay Loam, Silty Clay Loam	PR, BK, GR 0	1 Massive	61-75	0.30	0.40	0.40	0.50	0.50
4	Sandy Clay, Clay, Silty Clay	PR, BK, GR	2, 3	76-90	0.20	0.30	0.30	0.30	0.30
4A	Sandy Clay, Clay, Silty Clay	PR, BK, GR 0	1 Massive	91-120	0.15	0.20	0.20	0.20	0.20
5	Soil Types 2 – 4A	Platy	1, 2, 3	121+	0.10	0.15	0.15	0.15	0.15

[1]Source: Table 10-1. Colorado Reg. 43. June 2013. NOTE: Shaded areas require system design by a professional engineer. [2]Treatment levels are defined in Table 6-3 (Table 5 herein). [3]Higher long-term acceptance rates for Treatment Level 3N may be allowed for OWTS required to have a discharge permit, if the capability of the design to achieve a higher long-term acceptance rate can be substantiated. [3]Unlined sand filters in these soil types shall provide pathogen removal. Design shall conform to section 11.C.2.c, Unlined Sand Filters.

Table 10. Adjustments to soil treatment area size based on geometry, application method and infiltrative surface architecture (CDHPE, 2013).[1]

Type of Soil Treatment Area	**Method of Effluent Application to Soil Treatment Area**		
	Gravity	Dosed (Siphon or Pump)	Pressure Dosed
Trench	1.0	0.9	0.8
Bed	1.2	1.1	1.0
Type of Soil Treatment Area	Type of Storage/Distribution Media Used with TL 1		
	Rock or tire chips	Manufactured media other than chambers	Chambers
Trench or Bed	1.0	0.9	0.7

[1]Source: Table 10-2 and 10-3. Colorado Reg. 43. June 2013.

FOSNRS 1: The Florida Onsite Sewage Nitrogen Reduction Strategies (FOS-NRS) Study, Project Overview

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ABSTRACT

Onsite Wastewater Systems (OWS) serve approximately one-third of all households in Florida. The relative impact of OWS on total nitrogen loading varies from watershed to watershed with estimates ranging from below five to more than 20 percent. Regardless of the source, excessive nitrogen has negative effects on public health and the environment. There is widespread interest in the management of OWS to limit the nitrogen impacts in Florida and the nation. In discussions of the role of OWS in watershed protection, frequent questions concern the amount of nitrogen released from OWS, the attenuation of nitrogen after discharge to a drainfield, and the effectiveness, costs, and reliability of existing treatment technologies. To address the latter set of questions, the Florida Department of Health (FDOH) initiated and recently completed a comprehensive evaluation of the operation and management of the existing estimated 12,000 advanced systems in Florida, which are predominantly extended aeration systems. Treatment systems often had issues related to mechanical aeration where systems were turned off or the aerator was not working. Analysis of sampling results confirmed that such mechanical aeration failures had a direct effect on the nitrogen and cBOD5 removal performance levels of advanced systems. Complementing and following up on this study the Florida Department of Health has initiated the Florida Onsite Sewage Nitrogen Reduction Strategies (FOSNRS) Project to develop cost-effective, passive strategies for nitrogen reduction for OWS in Florida. The goal is to further evaluate the transport of nitrogen from OWS and to develop onsite nitrogen reduction systems that complement the use of conventional OWS, are cost effective and ecologically protective, and have reduced operation and maintenance costs. This paper provides a summary of the project tasks. The combined results of research on existing and potential future technologies will ultimately benefit Florida's approximately 2.6 million OWS owners by finding cost-effective nitrogen reduction strategies that will improve environmental and public health protection.

Onsite Wastewater Systems (OWS) in Florida serve approximately one-third of all households and number approximately 2.6 million. Most OWS in Florida are considered conventional systems, i.e. septic tanks with drainfields, which are designed to reduce human exposure to enteric pathogens. Conventional OWS have a limited capacity to reduce nitrogen concentrations in discharged effluent. Setbacks between OWS and potable drinking water wells are required to protect public health from exceedances of nitrogen maximum contamination levels as defined in the federal Safe Drinking Water Act. Programs within the Florida Department of Environmental Protection (FDEP) identify water bodies impaired by excessive nitrogen, establish targets for maximum nitrogen loads for these water bodies, and develop management action plans to restore them. The relative impact of OWS on total nitrogen loading varies from watershed to watershed, with estimates ranging from below five to more than 20 percent depending on contributions from other sources. Regardless of the source, excessive nitrogen has negative effects on public health and the environment. Exposed infants can develop methemoglobenia (blue baby syndrome) after ingestion of nitrates in drinking water. Increased amounts of nitrogen are a concern to Florida's springs and can cause eutrophication, which can lead to detrimental effects to sensitive aquatic ecosystems. There is widespread interest in the management of OWS to limit nitrogen impacts in Florida and the nation.

A technological approach to reducing excessive nitrogen from OWS is to require the use of an onsite system that provides enhanced sewage treatment. This approach has some history in Florida, where OWS are regulated by the Florida Department of Health (FDOH), and some form of better, or advanced, onsite sewage treatment has been required in several areas of the state for over a decade. Permitting categories for advanced systems include aerobic treatment units (ATU) certified to ANSI/NSF-Standard 40 (90%) and engineer-designed performance-based treatment systems (PBTS) (10%). Advanced systems require more maintenance and management than a conventional OWS. In Florida, owners of advanced systems must have a contract with a maintenance entity (ME), a company that is certified by a system manufacturer to perform maintenance inspections and ensure proper functionality. By Florida law they are also required to have a biennial operating permit issued by FDOH, annual inspections by FDOH, and two annual maintenance inspections by the ME.

In discussions of the role of OWS in watershed protection, frequent questions concern the attenuation of nitrogen after discharge to a drainfield and the effectiveness, costs, and reliability of existing treatment technologies. Consequently, FDOH initiated and recently completed a comprehensive evaluation of both the operation and management of the existing estimated 12,000 advanced systems in Florida. The study aimed to provide an assessment of what treatment options are available, how the systems are perceived, the effectiveness of the current management framework, what the operation and treatment performance for advanced systems is, what monitoring protocols are effective for consistent assessment, and what best management practices (BMPs) can be documented. This study was primarily funded through EPA's Nonpoint Source Pollution Section 319(h) grant program. A detailed review of a mostly random subset of identified systems was performed, as well as detailed field evaluations for 550 sites throughout Florida and effluent sampling of 350 advanced systems at different locations along the respective treatment trains.

Overwhelmingly, results showed that existing advanced treatment systems in Florida utilize an active mechanical extended aeration approach. The frequencies of various operational problems under the existing management approach were quantified. Common problems included issues related to mechanical aeration where systems were turned off or the aerator was not working. Analysis of sampling results confirmed that such mechanical aeration malfunctions had a direct effect on the nitrogen and cBOD5 removal performance levels of advanced systems. Overall, influent (pretreatment tank) total nitrogen concentrations found in the study (mid 40s mg/L) tended to be lower than in other recent studies (Lowe et al. 2009, Roeder 2011). Total nitrogen removal in operating systems was typically about a third.

In 2008 the Florida Legislature tasked FDOH with conducting a multi-faceted study of nitrogen removal by OWS, and to develop cost-effective, passive strategies for nitrogen reduction for OWS in Florida. Such treatment systems include those that use at most a single effluent pump with reactive media. The study is underway. The contract was awarded in January 2009 to a Project Team led by Hazen and Sawyer, P.C. Technical oversight is provided through a collaborative effort between FDOH, FDEP, and the FDOH Research Review and Advisory Committee (RRAC). The primary motivations for this study are the concerns about the potential environmental impacts of increased levels of nitrogen in water bodies. The objective of the FOSNRS Project is to evaluate and develop strategies to reduce nitrogen impacts from OWS regulated by FDOH. The goal is to further evaluate the transport of nitrogen from OWS and to develop onsite nitrogen reduction systems that complement the use of conventional OWS, are cost effective and ecologically protective, and have reduced operation and maintenance costs.

The project includes the following major tasks. Some of the tasks are complete and others are ongoing:

Task A – Technology Evaluation for Field Testing: Review, Prioritization, and Development (complete): Activities included a literature review, technology evaluation, prioritization of technologies to be examined during field testing, and further experimentation with approaches tested in a previous DOH passive nitrogen removal study. Controlled testing was done at a unique test facility to develop design criteria for new passive nitrogen reduction systems for testing at actual home sites.

Task B – Field Testing of Technologies and Cost Documentation (ongoing): Actual homes had top-ranked nitrogen reduction technologies installed, with documentation currently being collected on their performance and cost. The systems will have documented costs broken down by permitting, design, materials, construction, operation, and maintenance.

Task C – Evaluation of Nitrogen Reduction Provided by Soils and Shallow Groundwater (ongoing): Several field evaluations of nitrogen reduction in Florida soils and shallow groundwater were performed. Analysis of the data is ongoing. A simple planning model developed in Task D will use these data for calibration and validation. .

Task D – Nitrogen Fate and Transport Modeling (ongoing): The development of a simple fate and transport models of nitrogen from OWS is ongoing for use in the assessment, planning, and siting of OWS.

The contractor, in coordination with the RRAC and DOH, has successfully completed parts of Tasks A, B, C, and D, including literature reviews; ranking of nitrogen reduction technologies for field testing; design and construction of a test facility for further development of passive technologies; development of quality assurance documents for the test facility work, groundwater monitoring, field testing, and nitrogen fate and transport modeling; installation of full-scale passive nitrogen reducing systems at seven home sites; completion of numerous sampling events of passive systems at the test facility and field sites; design and construction of a soil and groundwater test facility; and field sampling of the soil and groundwater under OSTDS at residential homes throughout Florida and at the test facility.

Current efforts and work remaining includes: continuation and completion of field monitoring to determine the performance and cost of technologies at home sites. Studies of the fate and transport of nitrogen in the shallow groundwater; calibration and refinement of various nitrogen fate and transport models using field sampling results; and final reporting on all tasks with recommendations on onsite sewage nitrogen reduction strategies.

Results from the previously completed study on existing advanced OWS in Florida provide a point of comparison for results from the currently ongoing FOSNRS study. The combined results of research on existing and potential future technologies will ultimately benefit Florida's approximately 2.6 million OWS owners by identifying cost-effective nitrogen reduction strategies that will improve environmental and public health protection by identifying nitrogen reducing systems that protect groundwater and have reduced life-cycle costs and lower energy demands.

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FOSNRS 2: Passive, 2-Stage Biofilter Treatment Systems for Reduction of Nitrogen from OWS – Pilot Study Results

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ABSTRACT

In 2008, the Florida legislature provided funding to FDOH to develop cost-effective, passive strategies for nitrogen reduction that complement the use of conventional OWS, and the Florida Onsite Sewage Nitrogen Reduction Strategies (FOSNRS) Project was established. For the purposes of this project, passive systems were defined as treatment technologies that utilize no aerators, blowers, or mechanical devices other than a single pump and a reactive media for denitrification. The project started in early 2009 with an evaluation of nitrogen reduction options for OWS, followed by the development and testing of pilot-scale passive nitrogen reduction systems (PNRS). The PNRS pilot study was conducted over a period of 18 months and indicated that a two-stage biofiltration process was a relatively simple process that was effective in reducing nitrogen concentrations from primary treated wastewater effluent. The two stage process consisted of an aerobic, unsaturated porous media biofilter for nitrification, followed by an anoxic, saturated reactive media biofilter for denitrification. The unsaturated (Stage 1) biofilters were tested in both single pass and recirculation mode using either expanded clay, clinoptilolite or sand porous media. Anoxic (saturated Stage 2) biofilters were operated in upflow and horizontal modes using either elemental sulfur or lignocellulosic media as electron donors. Two-stage biofiltration, aerobic biofiltration followed by anoxic biofiltration, continuously achieved total nitrogen removals of over 95% from primary effluent in several of the pilot units. The pilot-scale testing results indicated that two-stage biofiltration appears to be a viable technology for nitrogen removal at individual home sites in Florida.

At the University of Florida Gulf Coast Research and Education Center located in Wimauma, Florida, a pilot test facility to evaluate two-stage biofiltration for nitrogen removal was established. The two-stage biofiltration systems consist of a first stage unsaturated media biofilter for nitrification, followed in series by a second stage saturated anoxic denitrification biofilter utilizing reactive media. Septic tank effluent (STE) is applied to the top of the first stage media, resulting in a downward percolation of wastewater over and through the media biofilter bed. The unsaturated pore spaces in the first stage media will allow air to reach microorganisms attached to the media surfaces, enabling aerobic biochemical reactions to occur. The significant target reactions are aerobic heterotrophic oxidation (by microorganisms that oxidize organic material and reduce biochemical oxygen demand), hydrolysis and ammonification (releasing ammonia), and nitrification (biochemical conversion of ammonia to nitrate and nitrite). Of particular interest are the organic and ammonia nitrogen concentrations in first stage effluent, and removal of oxidized nitrogen (nitrate and nitrite) by saturated Stage 2 biofilters.

MATERIALS AND METHODS

The PNRS study operated twenty-two pilot-scale biofilters over a period of 18 months to evaluate nitrogen removal from wastewater primary effluent as summarized in Table 1. The pilot test facility included unsaturated (Stage 1) biofilters in single pass and recirculation mode using expanded clay, clinoptilolite and sand media, saturated denitrification biofilters (Stage 2) in upflow and horizontal layout using lignocellulosic and sulfur media, and vertically stacked media designs. The twenty-two biofilters consisted of nine unsaturated Stage 1 biofilters, nine saturated Stage 2 biofilters, and four vertically stacked biofilter designs categorized into four groups as listed in Table 1.

Group A consisted of five two-stage systems which received primary effluent. The Group A systems were single pass Stage 1 biofilters directly connected to upflow Stage 2 denitrification biofilters. Target hydraulic loading to Group A Stage 1 biofilters was a surface loading of 3 gallons per square feet per day (gal/ft^2-day), which provided a 5.7 gal/ft^2-day surface loading to Group A Stage 2 biofilters (Table 1). The monitoring points included influent (primary effluent), Stage 1 effluent and Stage 2 (final effluent). Group B consisted of four Stage 1 biofilters with recirculation, which received primary effluent. Target hydraulic loading to Group B biofilters was a surface loading of 3 gal/ft^2-day forward flow and a recycle ratio of 3:1 of biofilter effluent to wastewater forward flow. This provided a 12 gal/ft^2-day total surface loading to the Group B biofilters (Table 1). The monitoring points included influent (primary effluent), recirculation tank effluent and Stage 1 effluent. Group C consisted of four Stage 2 biofilters which received composited Stage 1 effluents from Group B systems. Target hydraulic loading to Group C biofilters was a surface loading of 10 gal/ft^2-day. Monitoring points included the primary effluent, recycle tank effluent, and Stage 2 (final effluent). Dosing to Group A, B and C biofilters was once per hour (24 dose/day). Group D consisted of four biofilters with vertically stacked media which was unsaturated in the upper level and saturated at the lower level which are not further discussed within this paper.

Design and operation of the pilot biofilters is summarized in Table 1. Unsaturated (Stage 1) media included expanded clay, clinoptilolite and silica sand. Saturated (Stage 2) solid-state biofilter media included elemental sulfur and Southern Yellow Pine (lignocellulosic), with glycerol as a dosed liquid electron donor. The unsaturated biofilters (Stage 1) had two media depths: 15 and 30 inches with a larger media particle size occupying the upper one third of media depth and smaller particle size in the lower two thirds. Other media components included oyster shell and limestone as slow release alkalinity supply, and gravel.

RESULTS AND DISCUSSION

Performance data are presented in Figures 1 through 4 and are based in general on seven monitoring events. The primary effluent supplied to the pilot system had an average Total Nitrogen of 52.5 mg/L. Nitrogen in primary wastewater effluent is predominately in the form of reduced nitrogen. Total Kjeldahl Nitrogen (TKN) measures reduced nitrogen and is the sum of the two forms of reduced nitrogen, organic nitrogen and ammonia. Aerobic biofilters convert organic nitrogen to ammonia through ammonification and oxidize ammonia through nitrification. Effluent reduced nitrogen is therefore a good measure of Stage 1 performance. The reduced nitrogen in Stage 1 biofilter effluents are shown in Figure 1. Mean TKN levels vary from 2.4 to 4.0 mg/L, with standard deviations of approximately 1 mg/L indicating limited variability in effluent quality. The exception is the 30 inch clinoptilolite recycle biofilter (UNSAT-CL4), for which the high mean TKN and standard deviation were caused by one TKN result which was possibly a sampling artifact. Mean effluent ammonia nitrogen levels ranged from 0.01 to 0.5 mg/L, with many analyses at or below method detection limits. It is important to achieve low effluent ammonia in the Stage 1 biofilter because ammonia is not expected to be degraded in the anoxic environments of the saturated Stage 2 biofilters. Ammonia in Stage 1 effluent could pass through an anoxic Stage 2 biofilter and contribute to the total nitrogen in the final two-stage effluent. Organic nitrogen as well as ammonia in Stage 1 effluent could limit the removal efficiency of total nitrogen in the two-stage system. Verifying low reduced nitrogen levels in Stage 1 biofilter effluents is a first step in establishing effective two-stage nitrogen removal.

Saturated denitrification biofilters (Stage 2) contain electron donor media to remove nitrate and nitrite. Oxidized nitrogen is the sum of nitrate and nitrite (NOx), although nitrate typically dominates in biofilter effluents. Effective denitrification biofilters will have low levels of oxidized nitrogen in their effluent. Stage 2 biofilter effluent oxidized nitrogen levels are shown in Figure 2. Mean effluent oxidized nitrogen in sulfur biofilter effluents ranged from 0.04 to 0.11 mg/L with standard deviations of similar magnitude. Fluctuations in effluent oxidized nitrogen from the sulfur denitrification process were very limited. The glycerol biofilter provided similar oxidized nitrogen removal performance to the sulfur biofilters. Highly effective oxidized nitrogen removal was also achieved by the horizontal biofilter (LS1) that used Southern Yellow Pine sawmill waste as a lignocellulosic electron donor, producing mean effluent oxidized nitrogen of 0.02 mg/L. Two upflow lignocellulosic saturated (LS) biofilters exhibited incomplete oxidized nitrogen removal, with mean effluent oxidized nitrogen of 6.2 and 14.2 mg/L based on three monitoring events. Possible explanations for limited oxidized nitrogen removal in the two upflow LS biofilters include low media reactivity, insufficient retention time and biofilter design. The LS biofilter that achieved very effective oxidized nitrogen removal (LS1), which had a higher retention time, used similar lignocellulosic media, and other investigators have reported highly successful use of *Pinus radiata* (pine softwood) media in denitrification biofilters (Schipper et al., 2010). Overall, the pilot results verified denitrification biofilter designs that were highly effective in removing oxidized nitrogen.

The effluent from Stage 2 biofilters is the final effluent of a two-stage system. Stage 2 effluents include organic nitrogen, ammonia and oxidized nitrogen (nitrate plus nitrite). For a two-stage biofiltration system with effective first and second stages, effluent total nitrogen is dominated by dissolved organic nitrogen. Total nitrogen in denitrification biofilter effluents (Stage 2) are shown in Figure 3. All of these processes affect the total nitrogen removal efficiencies of two-stage biofiltration, which are shown in Figure 4. The nitrogen in Stage 2 effluents with sulfur media was uniformly dominated by dissolved organic nitrogen, as was effluent nitrogen from the glycerol fed Stage 2 biofilter. These had high total nitrogen removal efficiencies (Figure 4). Equivalent total nitrogen removal efficiencies of several lignocellulosic biofilters were limited by incomplete oxidized nitrogen removal, resulting in effluent nitrogen dominated by oxidized nitrogen. However, pilot testing results verified that several two-stage biofiltration designs could consistently achieve 95% total nitrogen removal.

CONCLUSIONS

The PNRS II study results over a period of 18 months indicate that the two-stage biofiltration process is effective in nitrogen removal from wastewater primary effluent. The unsaturated (Stage 1) biofilters in single pass and recirculation mode using expanded clay, clinoptilolite and sand media, consistently reduced ammonia nitrogen to less than 1 mg/L. Anoxic (saturated Stage 2) biofilters were operated in upflow and horizontal modes using elemental sulfur and lignocellulose (Southern Yellow Pine) as electron donors. Oxidized nitrogen (nitrate and nitrite) was consistently reduced to less than 1 mg/L in sulfur containing biofilters. Anoxic biofilters with lignocellulosic media did not consistently remove nitrate/nitrite under the conditions of this study. Two-stage biofiltration, aerobic biofiltration followed by anoxic biofiltration, continuously achieved total nitrogen removals of over 95% from primary effluent in several of the pilot units over the 18 month study. Two-stage biofiltration appears to be a viable technology for nitrogen removal at individual home sites in Florida. The results of this pilot study provided guidance for the design of full-scale systems at individual Florida home sites (FOSNRS 3 paper).

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Table 1. PNRS pilot test facility biofilters.

Biofilter Group	Description	Influent	Biofilter & Process Designations					
			Biofilter ID	Media Depth (inches)	Surface Loading Rate (gal/ft^2-day)	Biofilter ID	Media Depth (inches)	Surface Loading Rate (gal/ft^2-day)
A	Two Stage Biofilters Single Pass Stage 1 directly connected to Upflow Stage 2	Primary Effluent (septic tank effluent)	Stage 1 Single Pass / Stage 2 Upflow					
			UNSAT-EC1	15	3	DENIT-SU4	24	5.6
			UNSAT-EC3	30		DENIT-LS3		
			UNSAT-CL1	15		DENIT-SU3		
			UNSAT-CL3	30		DENIT-LS2		
			UNSAT-CL5	30		DENIT-LS4		
B	Two Stage Biofilters Stage 1 with effluent recycle to recirculation tank	Primary Effluent (septic tank effluent)	Recirculation Tank / Stage 1 Recirculating					
			RC1	None	N/A	UNSAT-SA2	30	12
			RC2	None		UNSAT-EC4	30	
			RC3	None		UNSAT-CL2	15	
			RC4	None		UNSAT-CL4	30	
C	Stage 2 Horizontal Saturated Biofilters	Composite Effluent from Group B Biofilters	DENIT-SU1	72	10			
			DENIT-SU2					
			DENIT-LS1					
			DENIT-GL1					
D	Vertically Stacked Media (In-Situ in-tank simulator) Single Pass Biofilters	Primary Effluent (septic tank effluent)	UNSAT-IS1	28	1.1			
			UNSAT-IS2					
			UNSAT-IS3		1.2			
		Nitrified Primary Effluent	UNSAT-IS4					

† EC= expanded clay; CL = clinoptilolite; SA= sand; LS = lignocellulosic; SU = elemental sulfur; GL = glycerol

Figure 1. Unsaturated biofilter effluent reduced nitrogen (Stage 1).
(Mean influent TN = TKN= 52.5 mg/L)

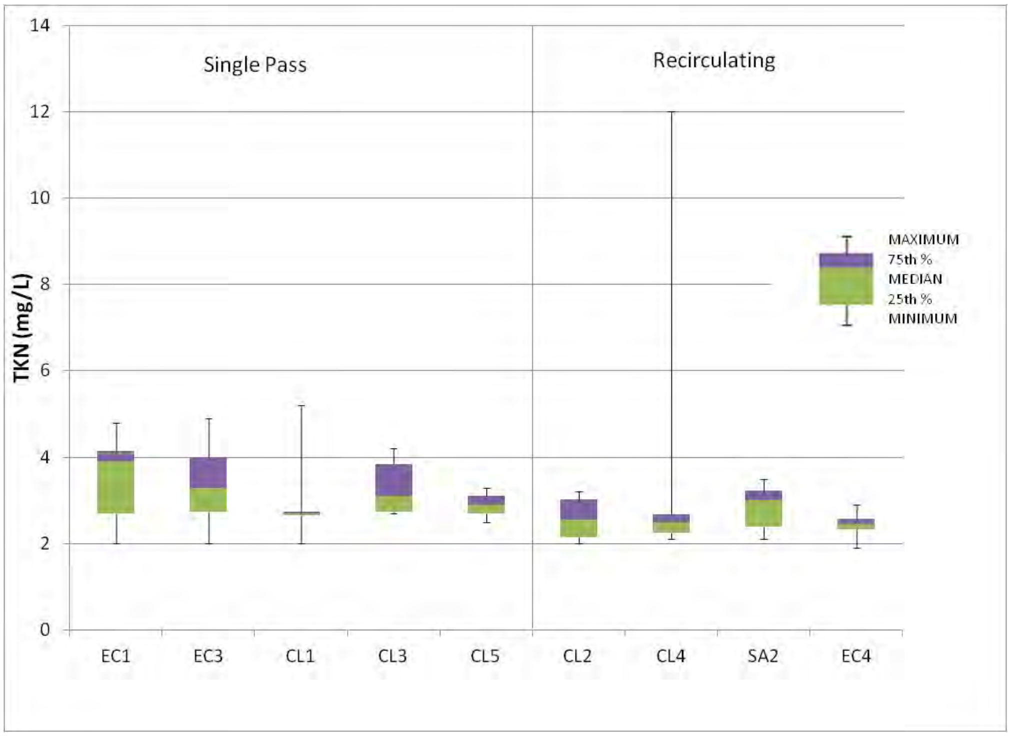


Figure 2. Saturated biofilter effluent oxidized nitrogen (Stage 2).

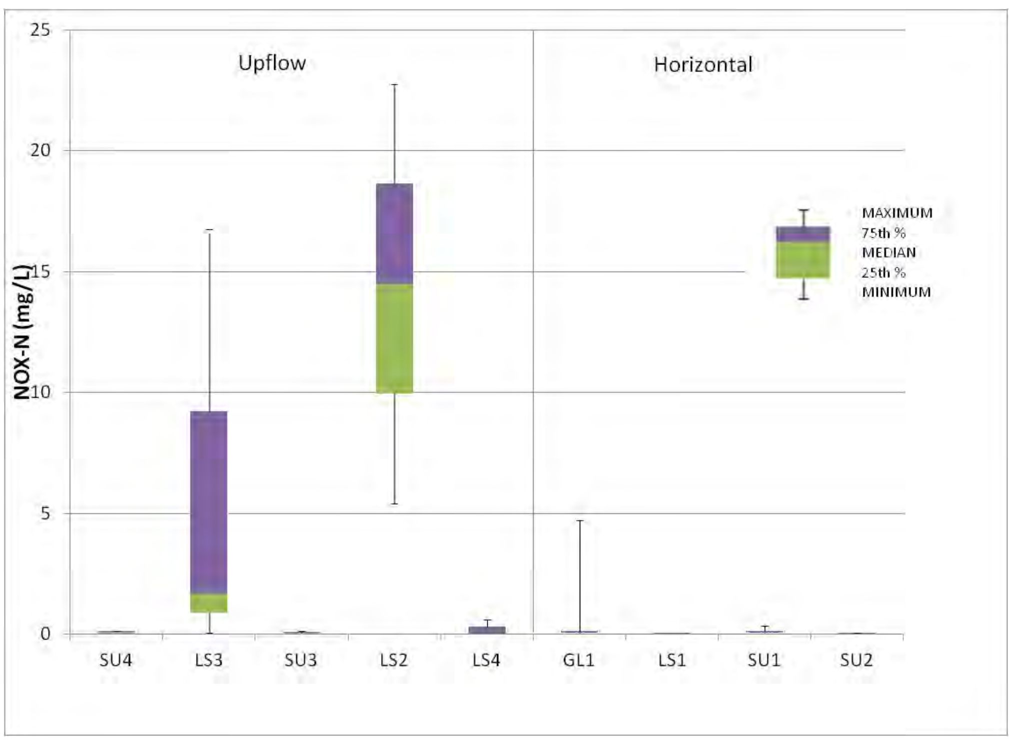


Figure 3. Saturated biofilter effluent total nitrogen (Stage 2).

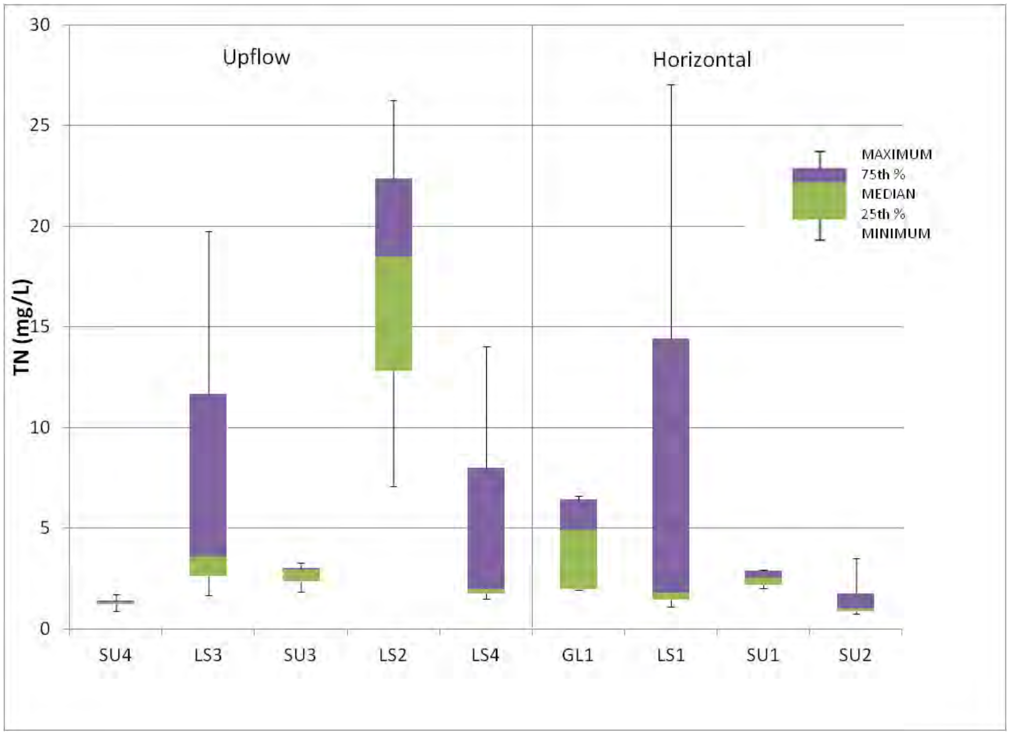
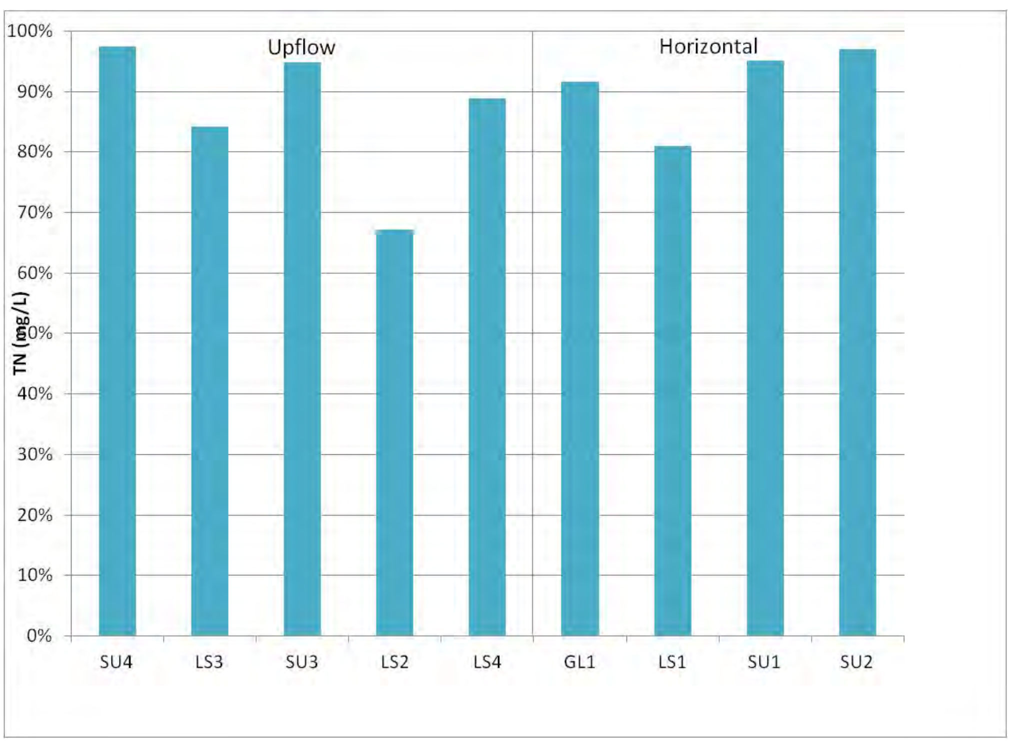


Figure 4. Equivalent total nitrogen removal efficiency.



FOSNRS 3: The Performance of a Full-scale 2 Stage Passive Biofilter System

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ABSTRACT

As part of the Florida Onsite Sewage Nitrogen Reduction Strategies (FOSNRS) project, passive nitrogen reduction systems (PNRS) were developed and pilot tested and are now being evaluated at homes in Florida. The goal of these systems is to reduce nitrogen inputs to watersheds where OWS have been identified as a significant source of nitrogen. Results from a full-scale PNRS installed at a 3 bedroom single family residence in Hillsborough County are presented here. This PNRS utilizes a two-stage passive biofiltration concept treating septic tank effluent (STE). The first stage provides ammonification and nitrification via a recirculating porous media biofilter. The second stage provides denitrification via an anoxic biofilter with reactive media. The system has been monitored over a 12 month period, receiving STE with an average total nitrogen concentration of 50 mg N/L. The Stage 1 biofilter with recirculation of nitrified effluent has consistently produced a nitrified effluent with ammonia N less than 3 mg N/L and a total nitrogen concentration averaging under 20 mg N/L. The second stage biofilter has consistently produced a final effluent with NO2-NO3 N concentrations below the method detection limit of 0.02 mg N/L. Residual ammonia nitrogen in the effluent from the Stage 1 biofilter passes through the Stage 2 biofilter resulting in an average TN concentration in the overall system effluent of 3.5 mg N/L, a reduction in total nitrogen of over 93%. While these are preliminary results, they suggest the potential to significantly reduce N input to sensitive watersheds from OWS.

Onsite Wastewater Systems (OWS) serve approximately one-third of all households in Florida. The relative impact of OWS on total nitrogen loading varies from watershed to watershed with estimates ranging from below five to more than 20 percent. Regardless of the source, excessive nitrogen has negative effects on public health and the environment. There is widespread interest in the management of OWS and the nitrogen impacts in Florida and the nation. For these reasons, the State has initiated the Florida Onsite Sewage Nitrogen Reduction Strategies (FOSNRS) Project. As part of the FOSNRS project, passive nitrogen reduction systems (PNRS) were developed and pilot tested and are now being evaluated at homes in Florida. Because of the flat topography common to the state, the definition of "passive" included the use of up to 1 pump as the only mechanical input to the system.

The goal of these systems is to reduce nitrogen inputs to watersheds where OWS have been identified as a significant source of nitrogen. Results from a full-scale PNRS installed at a 3 bedroom single family residence in Hillsborough County are presented here.

MATERIALS AND METHODS

The full-scale PNRS studied utilized the two-stage passive biofiltration concept at a home located in Hillsborough County, FL, just southeast of Tampa. The nitrogen reducing OWS for the 3 bedroom single family residence was installed in September 2012. Primary treated wastewater, or septic tank effluent (STE) from the home's existing septic tank is discharged to a two-stage treatment system consisting of a first stage unsaturated porous media recirculating biofilter for ammonification and nitrification, followed in series by a second stage saturated anoxic upflow porous media biofilter for denitrification. Flow to the system averaged 112 gallons per day during this evaluation period. A flow schematic of the system is shown in Figure 1. The system tankage consists of a 1,050 gallon two chamber concrete primary (septic) tank; 300 gallon concrete recirculation tank; 900 gallon concrete stage 1 unsaturated media biofilter; 300 gallon concrete pump tank; and 1,500 gallon two chamber concrete Stage 2 saturated media biofilter. The stage 1 unsaturated biofilter utilized an expanded clay porous media. Effluent from the stage 1 biofilter was pumped to the stage 2 biofilter and also recirculated back to the stage 1 biofilter at a ratio of approximately 3:1 recirculation flow R to forward flow Q. The stage 2 saturated anoxic biofilter consisted of two compartments, the first containing lignocellulosic media (saw mill waste) and the second containing elemental sulfur as electron donor reactive media for heterotrophic and autotrophic denitrification, respectively. Crushed oyster shell was added in the for alkalinity control in the stage 2 biofilter sulfur compartment. The denitrified treated effluent was discharged into the home's existing drainfield/soil treatment unit for final treatment and dispersal.

RESULTS AND DISCUSSION

The results presented here are based on system monitoring over a 12 month period, receiving 112 gpd of STE with an average total nitrogen (TN) concentration of 50 mg N/L. Figure 2 provides a graphic illustrating the mean water quality results through the treatment train. The Stage 1 biofilter with recirculation of nitrified effluent has consistently produced a nitrified effluent with ammonia N less than 3 mg N/L and a total nitrogen concentration averaging under 20 mg N/L. The Stage 1 recirculation scheme has resulted in an average 61 percent reduction in TN through the first stage alone. The second stage biofilter has consistently produced a final effluent with NO2-NO3 N concentrations below the method detection limit of 0.02 mg N/L. Residual ammonia nitrogen in the effluent from the Stage 1 biofilter passes through the Stage 2 biofilter resulting in an average TN concentration in the overall system effluent of 3.5 mg N/L, a reduction in total nitrogen of over 93%. Figure 3 provides a time series of the nitrogen data over the first year of operation. TN in the effluent from the two stage system consisted of approximately 40 percent organic nitrogen and 60 percent ammonia N. Thus, increasing the nitrification performance of the stage 1 biofilter could further enhance nitrogen removal from these systems, and investigations into this are underway.

Energy use by the system averaged 0.31 kWh per day, or 2.7 kWh per 1000 gallons treated. Operation and maintenance on the system has been minimal after an initial start-up period where system settings were established. There is no indication of any reduction in the reactive media (lignocellulosic or sulfur) levels after 1 year of operation.

CONCLUSIONS

Preliminary results of full-scale PNRS testing in the FOSNRS project indicate that consistent nitrogen reductions of over 90%, with total nitrogen effluent concentrations under 5 mg N/L may be possible with a two-stage biofilter system as described here. While these are preliminary results, they suggest the potential to significantly reduce N input to sensitive watersheds from OWS. Six additional full-scale PNRS are currently under early stages of evaluation, and results from these systems will provide key additional data regarding PNRS performance.

Figure 1. Flow schematic for the passive nitrogen reduction system (PNRS) installed in Hillsborough County, Florida

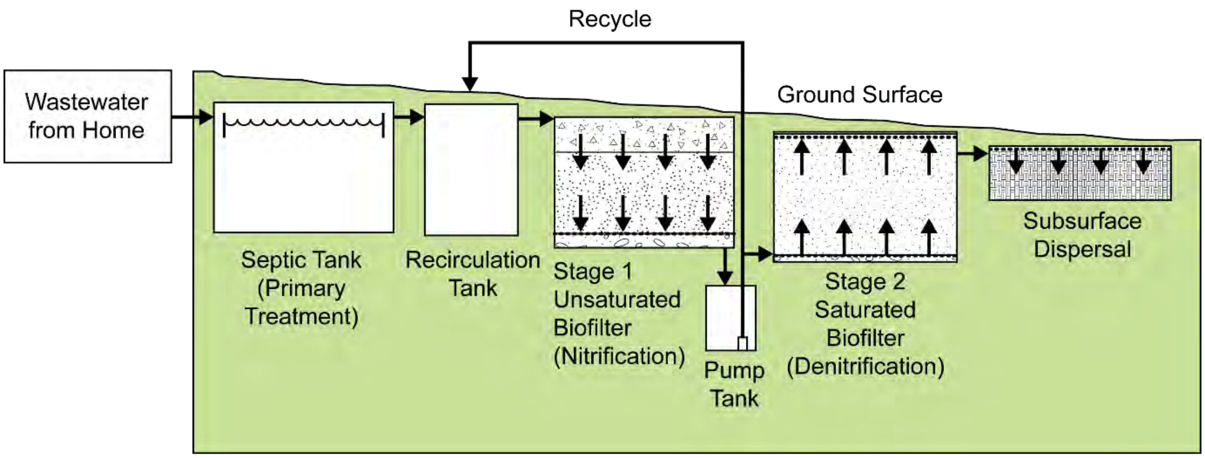
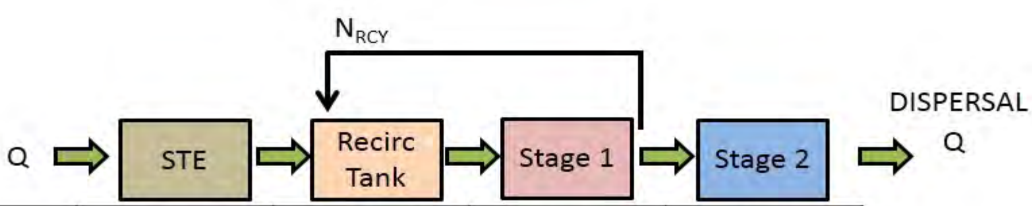
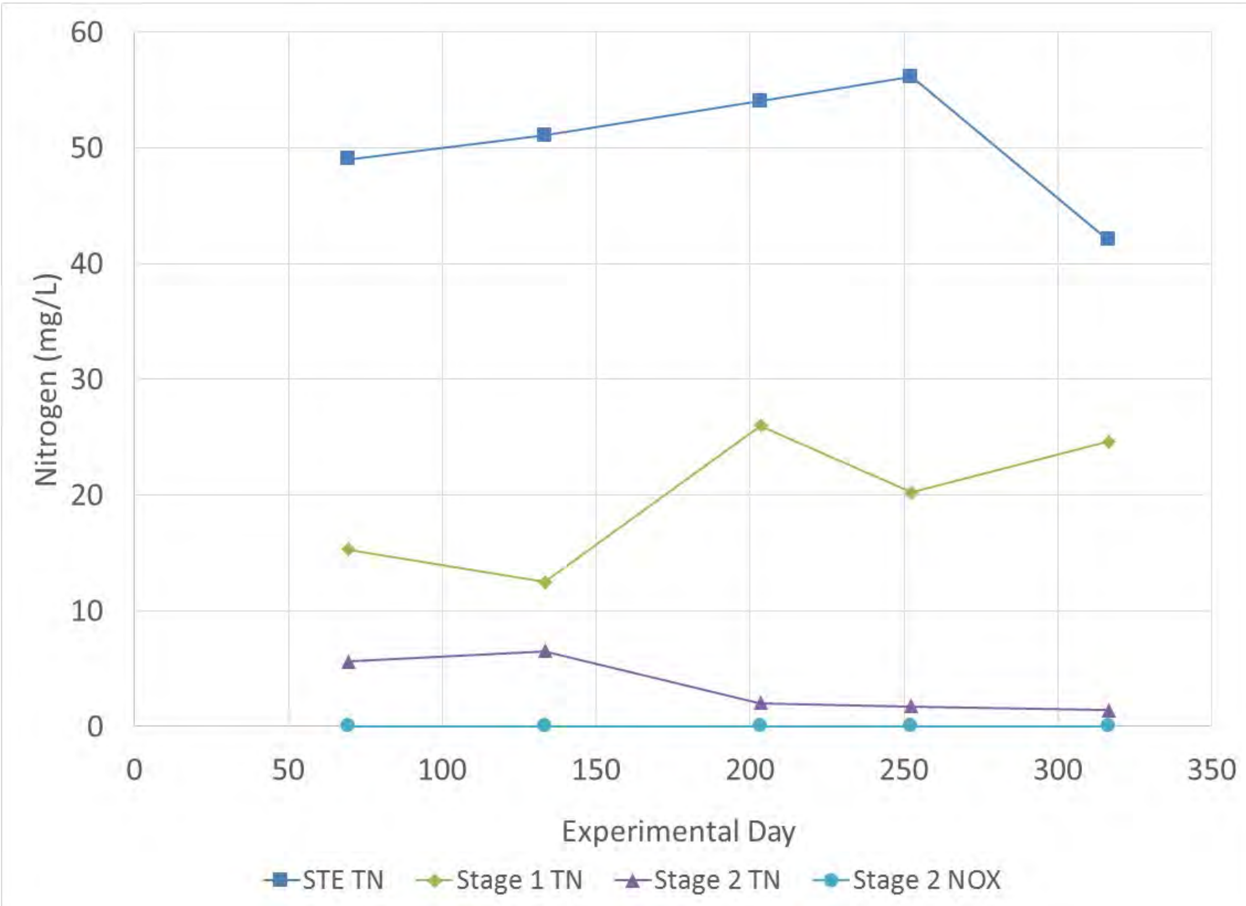


Figure 2. Mean water quality results over first year of PNRS operation



		Septic tank effluent	Recirc tank effluent	Stage 1 effluent	Stage 2 effluent
n		5	5	5	5
$CBOD_5$ mg/L	mean	105.6	25.2	15.2	67.6
TKN mg N/L	mean	50.4	12.8	3.1	3.4
NH_3 mg N/L	mean	41.6	9.0	0.9	2.2
NO_x mg N/L	mean	0.05	6.1	16.7	0.02
TN mg N/L	mean	50.5	18.9	19.7	3.5
Sulfate mg/L	mean	83.4	not analyzed	not analyzed	192
Fecal Coliform (Ct/100 Ml)	geomean	115,416	38,350	166	53

Figure 3. Time series of nitrogen data for PNRS over first year of operation.



Water and Nitrogen Balance for Mounded, Drip Irrigation Systems Receiving Septic Tank Effluent.

Gurpal Toor, University of Florida

ABSTRACT

Critical for understanding OWS performance is the water and nitrogen (N) budget below a soil treatment unit (STU), or drainfield. The objective of this FOSNRS project investigation was to determine the mass balance of water and N below mounded drip irrigation STUs and determine how the data could be scaled up to site and regional impacts. To achieve this, bench scale mound systems were established at the FOSNRS test facility at the UF Gulf Coast Research and Education Center (GCREC). Three small mound systems were constructed in treated wood boxes that were 5 ft. long, 3 ft. wide and 3 ft. high with 1:1 side slope (hereafter referred to as micro-mounds). Each micro-mound was 33 inches high from bottom to top and included 3 inches of washed gravel and sand at the base; followed by 12 inches of natural soil; 12 inches of sand; a single drip line with 3 emitters, and 6 inches of sand on top of the drip line. St. Augustine grass (19.2 square ft) was planted on the top and sides of each micro-mound. A hole was drilled at the bottom of each micro-mound box to which a floor drain strainer was attached to collect the percolate. Each micro-mound received 2.4 gallons per day (gpd) of septic tank effluent (STE, equivalent to maximum allowable rate 0.8 gpd/square ft. for Florida sandy soils), dosed 6 times per day via the drip emitters. Each micro-mound was instrumented with 10 multi-probes (CS 650, Campbell Scientific Inc.) to measure volumetric moisture content, electrical conductivity (EC), and soil temperature in different layers and sides. Results show that major input of water during January through September 2013 was from STE (52%) and rainfall (47%), while major water output was percolate (47% of total input) and evapotranspiration (29% of total input). About 24% of the added water (STE and rainfall) was stored in the mound. Concentrations of total N in the percolate, sampled during 50 events, from January through September ranged from 2 to 60 mg N/L, with nitrate-N being the dominant N fraction, followed by dissolved organic N. As the study is ongoing, the mass balance of N has not yet been computed. However, preliminary data shows that as expected, the major input of N is from STE (98.3%), followed by rainfall (1.7%), while about 30% of applied N was recovered in the percolate and about 13% of N was removed by plant uptake. This implies that about 57% of applied N is either stored in the soil and/or otherwise lost (denitrification, anammox) from the micro-mounds. Future plans include destructively sampling the micro-mounds in early 2014 to determine N stored in soil to update the N mass balance and calculate the proportion of N lost. This presentation will discuss the mass balance of water and nitrogen in micro-mounds and how the data from this study can be scaled up to present water and N budget for mounded OWS. Further, this data can be scaled up to a regional level to estimate the amount of water and N loading to groundwater below mounded STUs in sandy soils.

Quantifying Rates of Denitrification and Microbial Activity in the Biozone and Shallow Subsurface within Soil Treatment Units Used for Wastewater Reclamation

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ABSTRACT

A key constituent of concern affecting soil treatment unit (STU) design is nitrogen due to its potential transport to groundwater and adverse effects on human and ecosystem health. Research was completed at the Colorado School of Mines to evaluate denitrification in STUs and to what extent the N species in the effluent affects the potential and expressed rates. Under laboratory conditions at the Mines Park Test Site in Golden, Colorado four columns were packed with soil classified as a Seffner fine sand obtained from the Florida Onsite Sewage Nitrogen Reduction Strategies (FOSNRS) test facility in Hillsborough County, Florida. All four columns were dosed twice daily to yield a hydraulic loading rate of 2 cm/d with two columns receiving septic tank effluent (STE) and two columns receiving nitrified intermittent sand filter (ISF) effluent. Denitrification rates (DNR) were measured after 10 weeks and 28 weeks of operation. DNR measurements were made to determine 1) the representative DNR (DNR_R) under actual conditions in the column using static core acetylene inhibition and 2) the potential DNR (DNR_P) under optimal conditions using denitrification enzyme activity. In addition, quantitative polymerase chain reaction (qPCR) was conducted on soil samples to enumerate the presence of genes associated with denitrification (*nirS*, *nirK*, *nozZ*). Results from this research indicate that rates of denitrification in native soil can be substantially increased by effluent application and can be elevated to higher levels in soil receiving STE compared to nitrified ISF. The highest recorded levels of representative denitrification rates, potential denitrification rates and denitrification genes *nirS*, *nirK*, and *nozZ* were all documented at a depth of 0-1 cm below the infiltrative surface of a column receiving STE.

INTRODUCTION

Soil treatment unit (STU) operations are commonly used to achieve tertiary treatment with natural disinfection within onsite and decentralized wastewater treatment systems. The primary removal process for total nitrogen entering the subsurface via a STU is denitrification. Compilation and analysis of rates of denitrification reported in the scientific literature has documented a range of rates that vary by five orders of magnitude (Fig. 1) (Tucholke, 2007). Efforts to predict nitrogen fate in and below a STU has identified the large variability in denitrification rates as a major contributor to uncertainty in quantifying nitrogen fate. The nitrogen fate model STUMOD (Soil Treatment Unit Model) (Geza et al., 2009), was originally developed to predict the fate and transport of nitrogen in a soil treatment unit. STUMOD is an analytical model that is implemented through a spreadsheet. The model calculates nitrogen species concentrations with depth in the soil profile and the fraction of nitrogen remaining with depth. Sensitivity analyses conducted using STUMOD have identified that the model outcomes are particularly sensitive to the input parameter value selected for the rate of denitrification (Geza et al., 2009; Heatwole and McCray, 2007). This paper describes the research that was completed to develop and assess a method for quantifying representative and potential rates of denitrification in soil subject to wastewater application such as occurs in a STU.

MATERIALS AND METHODS

The soil used in this research was collected in Hillsborough County at the University of Florida's Gulf Coast Research and Education Center near Wimauma, Florida. The soil was obtained from the Florida Onsite Sewage Nitrogen Reduction Strategies (FOSNRS) test facility located at the site and was classified as Seffner Fine Sand with sand, silt, and clay fractions of 95%, 1%, and 4% (dry wt.%), respectively, pH = 5.0, and organic matter content = 0 to 0.05 dry wt.%. The Seffner Fine Sand was shipped to CSM where under field laboratory conditions at the Mines Park Test Site in Golden, Colorado, four columns were packed to an average dry bulk density of 1.57 g/cm^3, within the range of 1.35 to 1.70 g/cm^3 that was reported for the soil bulk density under field conditions. Clear acrylic columns (16-cm diameter by 60-cm long) were packed with a 60-cm layer of the Seffner fine sand. After packing, the average saturated hydraulic conductivity (K_{SAT}) of the four soil columns was 359 cm/d (coeff. var. = 22%)

The columns were established in a field laboratory at Mines Park Test Site. The columns were cloaked with black plastic to maintain darkened conditions and the average temperature was 17.7°C. All four of the soil columns were dosed with effluent twice daily to yield a daily hydraulic loading rate (HLR) of 2 cm/d. Two columns were dosed with domestic septic tank effluent (STE) and two columns were dosed with nitrified, intermittent sand filter effluent (ISF) (Fig. 1). The STE was generated from an apartment building that housed student families and was located near the Mines Park Test Site. This same effluent had been used for a variety of laboratory and field experiments over a period of more than a decade (Siegrist *et al.*, 2013). Within the laboratory, the STE was treated in two sand columns (that mimicked an intermittent sand filter (ISF) with a HLR of 5 cm/d) to generate an ISF effluent that was used to dose two of the soil columns. During operation of the soil columns, monitoring included the HLRs and composition of the effluents applied to the columns, the occurrence and duration of ponding on the infiltrative surface, and the volume and composition of the percolates exiting the columns. Monitoring of the effluents applied to the soil columns and the percolates exiting the columns included weekly collection of sampling and analysis of pH, alkalinity, chemical oxygen demand, total nitrogen, nitrate and ammonia. Dissolved organic carbon (DOC) and UV absorbance (SUVA) were measured at weeks 4, 6, 8 and 28.

After 10 weeks of operation two of the four soil columns were carefully disassembled and dissected to enable measurement water filled porosity (WFP), potential rates of denitrification (DNR_P), and representative rates of denitrification (DNR_R) at multiple depths below the infiltrative surface. One column had been dosed with STE for 10 weeks and one column had been dosed with ISF effluent for 10 weeks. DNR rates were measured at depths ranging from 8-60 cm below the infiltrative surface. DNR_R rates were measured using the static core acetylene inhibition method and DNR_P rates were measured using the denitrification enzyme activity method (Farrell, 2013; Gillam et al., 2008; Groffman et al., 2006). Soil samples for qPCR analysis were aseptically collected and placed into 3-mL conical vials and stored in a cryogenic freezer (-80°C) prior to shipment to Ireland for analysis. At the University of Galway, collaborators Barrett et al. developed and implemented the methods for DNA extraction and gene targeting using quantitative PCR. (Barrett et al., 2013).

After 28 weeks of operation the same process of dismantling and dissection was repeated in the two remaining soil columns. During the second round of analysis samples for WFP, DNR_R, DNR_P and qPCR were isolated at depths ranging from 0-60 cm below the infiltrative surface (bis). The primary difference between the first sample collection event at 10 weeks and the second event at 28 weeks is that during the second sample collection, sampling and analysis was also focused on the biozone at and immediately below the soil infiltrative surface (0-1 cm bis).

RESULTS AND DISCUSSION

A summary of the water quality parameters for the effluent applied to each column is provided in Table 1. The average concentration of total nitrogen in the STE and ISF effluent was 41.7 and 34.9 mg-N/L, respectively. After 10 weeks of operation, mass balance analysis revealed that the columns were achieving a very low total N removal (0-16%). Samples collected after 28 weeks of operation also identified a statistically insignificant removal of nitrogen within the same range. The low N removal was consistent with expectations and model predictions made using STUMOD (Farrell, 2013).

The average concentration of DOC in the STE effluent (10.4 mg-C/L) was over two times higher than the DOC concentration in the ISF effluent (4.7 mg-C/L). The average DOC concentrations in the STE and ISF column percolates were closer in value (6.4 mg-C/L in STE versus 4.4 mg-C/L in ISF). Based on DOC and SUVA measurements, in comparison to the ISF effluent the STE provided a richer source of biodegradable organic matter. The richer source of organic matter may have contributed to the development of biological clogging at the soil infiltrative surface in the columns dosed with STE. The infiltrative surface of the columns dosed with STE had a visible black colored biomat that caused STE to pond for two hours after dosing. In contrast, one minute after a dose was applied to the ISF columns no effluent ponding was observed.

The sampling event conducted after 10 weeks of operation had measured WFP values that varied from 37% (v/v) at 10-22 cm bis to 70% at 47-59 cm bis. For the STE column the average DNR_R rate was 0.0020 mg-N/d per L of column pore volume (PV), which was only ~5% of the DNR_P that was measured at 0.037 mg-N/d per L PV. For the ISF column, the average DNR_R was below reporting levels and the DNR_P was 0.0026 mg-N/L PV. DNR_P rates were measured in the native Seffner fine sand as a control and the rate was zero. Therefore, effluent addition had increased the DNR_P in both columns.

The low DNR rates documented during the first sampling event prompted a more detailed analysis of how denitrification rates are used by the nitrogen fate model STUMOD. An important function of the model is its ability to adjust the rate of denitrification based on the dynamic characteristics in a STU. STUMOD adjusts a maximum rate of denitrification according to how water filled porosity, temperature, organic carbon and the concentration of nitrogen change as a function of depth (McCray et al., 2010). As each of these parameters vary in response to operational conditions such as HLR, climate and water table depth, a factor between zero and one is applied to the maximum denitrification rate. STUMOD uses the following equation to adjust the maximum rate of denitrification ($\mu_{max,den}$) to obtain the effective rate of denitrification (S_{den}):

$$S_{den} = f_z \cdot f_{sw} \cdot f_t \cdot \mu_{max,den} \left(\frac{C_{NO3}}{K_{m,NO3} + C_{NO3}} \right) \qquad [1]$$

where f_z is a factor for organic carbon availability, f_{sw} accounts for WFP, f_t accounts for temperature, C_{NO3} and $K_{m,\ NO3}$ account for the concentration of nitrate using Monod kinetics. When using STUMOD to model a sandy soil receiving STE at a HLR 2 cm/d, the adjusted rate of denitrification is the highest at a depth of 0-2 cm bis. Figure 2 demonstrates how STUMOD adjusts the effective rate of denitrification for this scenario.

The sampling event conducted after 28 weeks of operation repeated the WFP and DNR measurements at the same depths measured after 10 weeks. However, samples from the biozone (0-1 cm bis) were also included in response to the STUMOD prediction of elevated rates of denitrification in that zone.

DNR measurements made depths at >8 cm bis revealed average rates similar to those observed at 10 weeks. WFP values varied from 31% (v/v) at 10-22 cm bis to 73% at 47-59 cm bis. The WFP values at these depths were also similar to those recorded at 10 weeks. WFP values were 50% (v/v) at 0-1 cm bis in the STE column (biozone) and 41% (v/v) at 0-1 cm in the ISF column. The elevated levels of WFP near the infiltrative surface of the soil columns may have played an important role in creating anoxic conditions to sustain denitrification.

DNR measurements at and close to the infiltrative surface were significantly higher than rates measured deeper in the columns (i.e. at >8 cm depth bis) (Fig. 3). For the 0-1 cm depth in the STE column, the average DNR_R was 0.21 mg-N/d per L PV and the average DNR_P was 1.35 mg-N/d per L PV. For the 0-1 cm depth in the ISF column, the DNR_R was below reporting levels and the average DNR_P was only 0.033 mg-N/d per L PV. Illustrating the effects of effluent quality, for the ISF column (where nitrified sand filter effluent was applied) the DNR_P measured at 0-1 cm depth bis was only ~2% of the DNR_P measured at the same depth in the STE column. The rates measured in this research are compared to values reported in the literature as shown in Fig. 4.

Soil samples were obtained for qPCR analysis from approximately 0.5 cm above the location of all DNR_P samples collected at both 10 weeks and 28 weeks of operation (Fig. 5). The gene copy concentrations of the denitrifier genes *nirS*, *nirK* and *nosZ* were similar between all STE and ISF column samples excluding the sample at 0-1 cm bis of the STE column. The average concentrations of *nirS*, *nirK* and *nosZ* genes at 0-1 cm bis of the STE column were 1.9×10^8, 2.9×10^6 and 5.8×10^7, respectively. These concentrations were one to three orders of magnitude higher than the average *nirS*, *nirK* and *nosZ* levels reported for all the other samples. The average gene copy concentrations for all the other STE and ISF column samples were 7.8×10^5, 1.8×10^4 and 1.2×10^6, respectively. The highest concentration of gene copies for *nirS*, *nirK* and *nosZ* were all recorded at 0-1 cm depth bis in the STE column.

CONCLUSIONS

The results of this research reveal that rates of denitrification in native soil can be substantially increased by effluent application and can be elevated to higher levels in soil receiving STE compared to nitrified ISF. This project used numerous methods of investigation to demonstrate that, in a Seffner fine sand with a HLR of 2 cm/d operated for 10 or 28 weeks, denitrification occurs at the highest levels in the biozone (0-1 cm bis) of a fine sandy soil loaded with STE. Considering STE has effluent with nitrogen in the form of ammonia, the microbial community in the biozone must contain nitrifiers and denitrifiers coexisting near the infiltrative surface. This is contrary to the conceptual model that there is a nitrification zone (e.g., 0 to 15 cm depth bis) followed by a denitrification zone (e.g., at a depth >15 cm bis). This project highlights the importance of the development of the biozone to enhance wastewater treatment in a STU. During this research the columns receiving nitrified ISF effluent did not develop a biozone that was visually distinct. The lack of a biozone may impact the potential for soil to develop and sustain a microbial community capable of the highest levels of nitrogen removal.

ACKNOWLEDGEMENTS

This work was funded in part by the Florida Department of Health through a contract with Hazen and Sawyer. The authors would like to thank Damann Anderson of Hazen and Sawyer for his interest in and critique of the research methods and results. The authors acknowledge Dr. John Spear of CSM for his advice concerning the experimental methods, and Teena Stockert and Dr. Dave Smart of the Department of Viticulture and Enology at University of California - Davis for their guidance and laboratory assistance with quantifying nitrous oxide concentrations. Aaron Clubb, graduate student at CSM provided significant assistance related to experimental design and implementation. Dr. Laurence Gill of Trinity College in Dublin, who was on sabbatical at CSM during the study period, was instrumental in facilitating the collaboration with researchers at the National University of Ireland - Galway. Kathryn Kilroy, Ph.D. student at the National University of Ireland - Galway, assisted with completion of the qPCR analyses.

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Table 1. Composition of effluents applied to the soil columns.[1]

Parameter	Units	Septic tank effluent				Intermittent sand filter effluent			
		Average	S.E.	Range	*n*	Average	S.E.	Range	*n*
pH	-	7.7	0.1	7.1 - 7.9	9	5.6	0.2	4.8 - 6.4	10
Alkalinity	mg $CaCO_3$ / L	181	6.5	164 - 220	9	3	1.7	0 - 14	10
COD	mg / L	128	21	56 - 262	9	8.5	1.7	0 - 16	9
TN	mg N / L	41.7	1.5	33 - 49	9	34.9	1.2	29 - 41	10
NO_3	mg N / L	1.2	0.3	0.4 - 3.4	9	31.4	2.8	15.0 - 39.4	10
NH_4	mg N / L	39.6	2.5	21.4 - 46.2	9	2.7	0.4	0.4 - 4.3	9
DOC	mg C / L	10.4	1.0	8.7 – 12.2	3	4.7	0.8	3.5 – 6.3	3
SUVA[2]	L / mg C	1.4	0.1	1.3 - 1.5	3	1.4	0.1	1.2 – 1.6	3

[1]STE data are for weeks 2 to 10; ISF data are for weeks 1 to 10. [2]SUVA = specific UV absorbance.

Figure 1. Experimental system setup, flow regime and soil column detail.

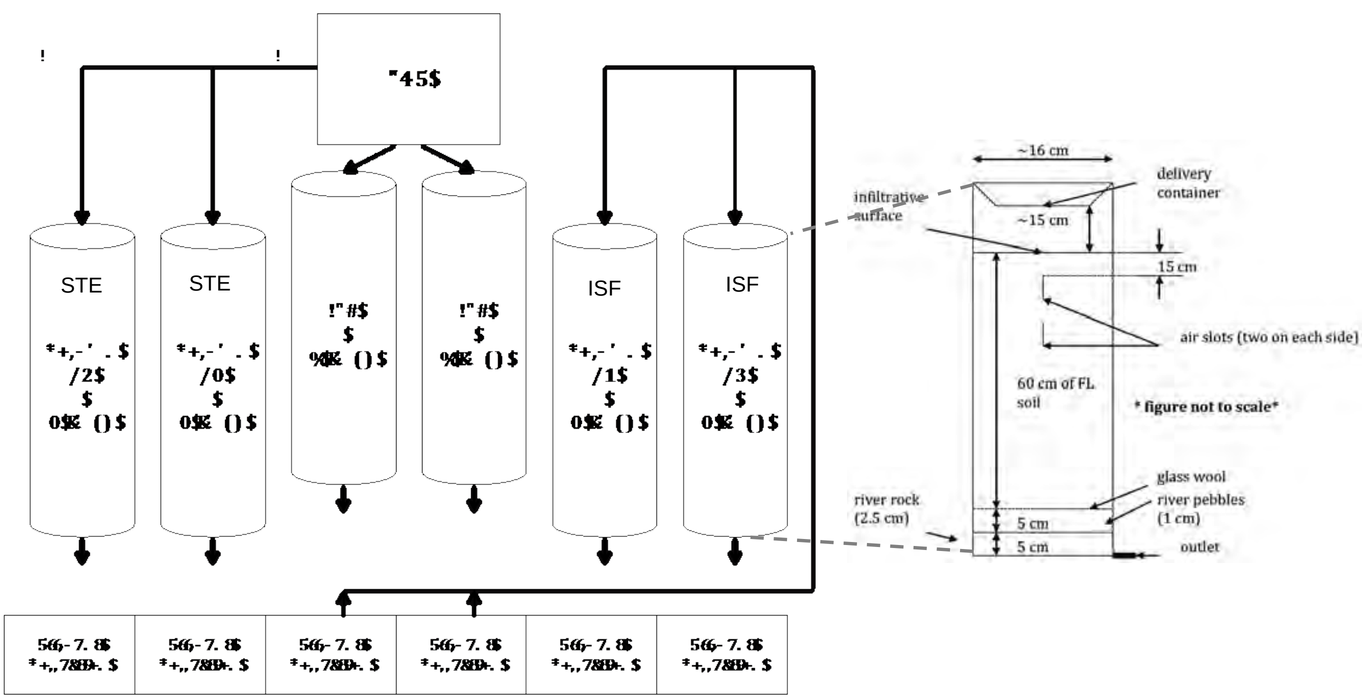


Figure 2. Adjustment of a maximum denitrification rate for sandy soil receiving STE at a HLR of 2 cm/d as implemented in STUMOD.

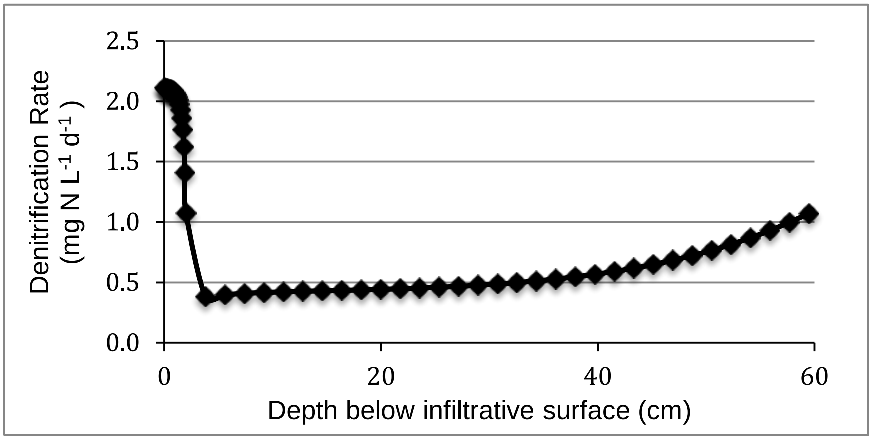


Figure 3. DNR_P and DNR_R rates measured at different depths below the infiltrative surface in a soil column packed with Seffner Fine Sand after receiving STE at a HLR of 2 cm/d for 28 weeks.

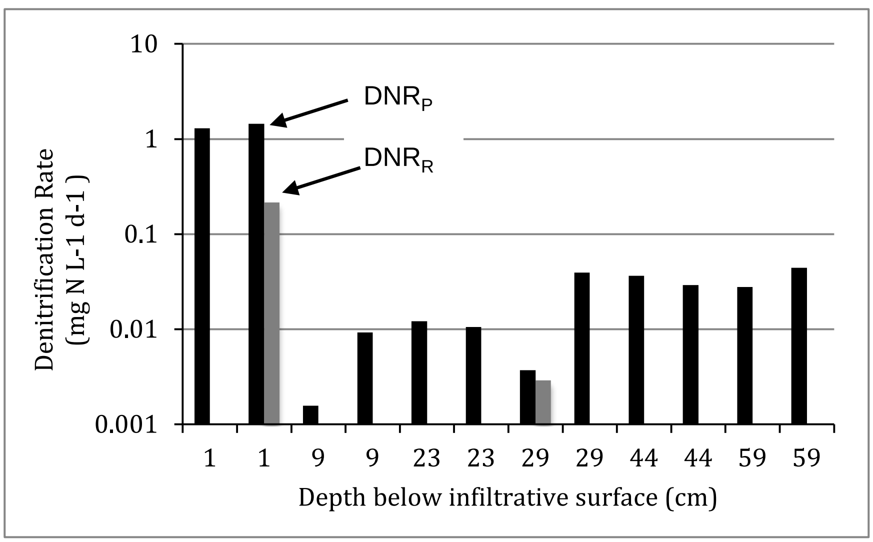


Figure 4. Comparison of the DNR measured in this research to the rates reported in published literature sources (Tucholke, 2007).

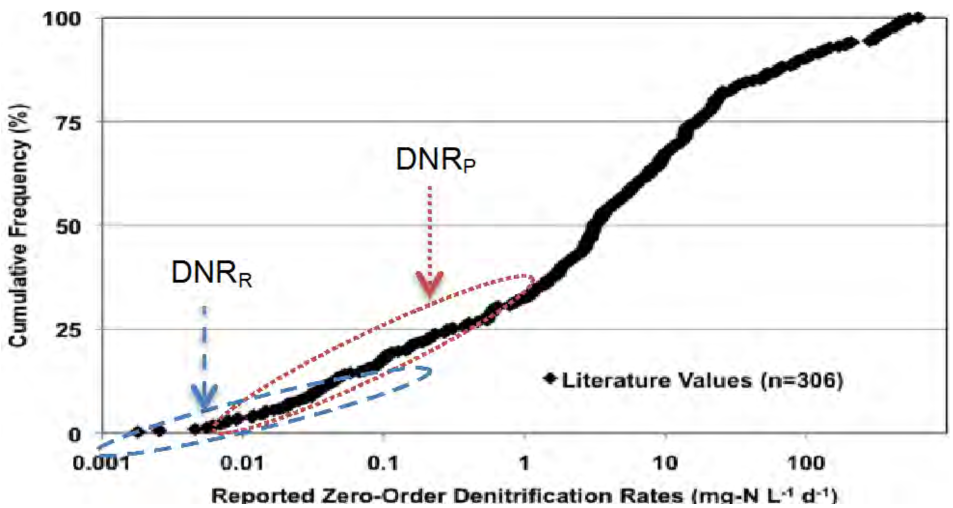
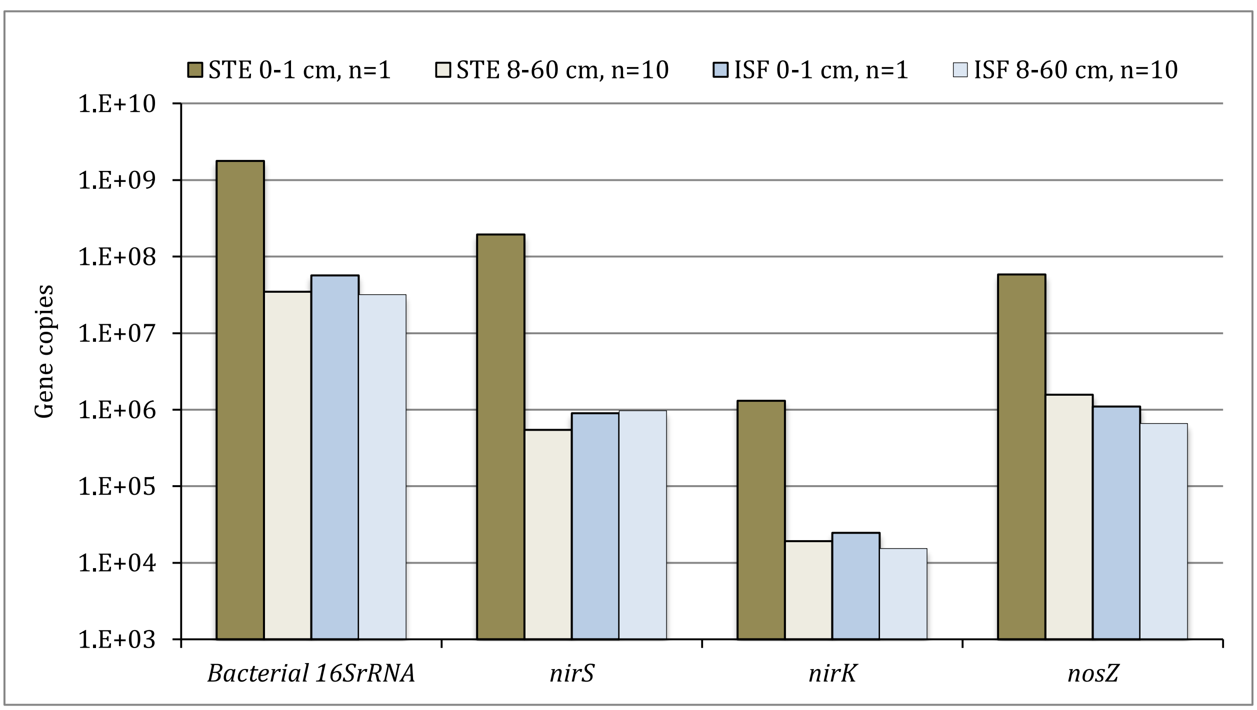


Figure 5. Results of qPCR analyses on samples collected at different depths below the infiltrative surface in soil columns packed with Seffner Fine Sand after receiving STE or ISF effluent at a HLR of 2 cm/d for 10 weeks and 28 weeks of operation.



STUMOD-FL - A Tool for Predicting Fate and Transport of Nitrogen in Soil Treatment Units in Florida

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ABSTRACT

A practical modeling tool to evaluate the fate and transport of nitrogen in Onsite Wastewater Systems (OWS) subsurface is being developed by the Colorado School of Mines (CSM) in collaboration with Hazen and Sawyer and Florida Department of Health (FDOH) as part of the Florida Onsite Sewage Nitrogen Reduction Strategies (FOSNRS) project. The modeling includes fate and transport of nitrogen both in the saturated and unsaturated zones building off of the Soil Treatment Unit Model (STUMOD) developed at CSM through support from the Water Environment Research Foundation. This new version, STUMOD-FL, developed for the FOSNRS project, is a simple to use spreadsheet tool designed to simulate the physical processes that control the movement of wastewater constituents through the vadose zone. An aquifer module for fate and transport of nitrate is being developed to link to STUMOD-FL. STUMOD-FL is based on fundamental principles of water movement and contaminant transport using an analytical solution to calculate suction and soil moisture profiles in the vadose zone and a simplification of the general advection dispersion equation. STUMOD-FL is tailored to Florida-specific soil and climate conditions and includes default model parameters representing dominant soil properties found in Florida. However, STUMOD-FL allows user-specified input and can be calibrated to site-specific data. STUMOD-FL accounts for the effect of nutrient uptake and variable water tables on nitrogen removal and incorporates up to four soil layers including the biomat. The nutrient uptake is driven primarily by plant nutrient demand, nutrient availability and rate of evapotranspiration. Plant nutrient demand is a user input; nutrient availability or soil water concentration and evapotranspiration rates are calculated by the model. Both nitrate and ammonium species are assumed to be equally available to plants. Nitrogen concentration reaching the water table depends on the thickness of the unsaturated zone. Users can input a known distance to water table if available or use a water table fluctuation module integrated to STUMOD-FL. The water table fluctuation module calculates seasonal variability of water in response to precipitation, evapotranspiration and aquifer properties. Nitrogen species concentrations and the fraction of total nitrogen reaching the aquifer, and mass flux is estimated by STUMOD-FL at user specified soil depths based on user input data including hydraulic loading rate, sorption, nitrification and denitrification rates. The effect of soil moisture content (a surrogate for redox conditions) is considered both in nitrification and denitrification reactions. STUMOD-FL predictions are being compared to controlled pilot-scale test site data on nitrogen fate and transport under a variety of typical operating conditions. Model outputs provide

insight into the behavior of soil treatment and quantitative estimations of nitrogen removal as affected by a range of conditions.

INTRODUCTION

A modeling tool intended for estimating loading to groundwater must consider transport and transformation in the vadose zone, because the N transformations that occur in this zone have considerable influence on the mass-flux input into an underlying aquifer. STUMOD-FL is a practical tool that accounts for most relevant processes and has the capacity to predict fate and transport of nutrients reasonably well with minimal data input. For the unsaturated zone, we assume predominantly vertical flow and contaminant transport by advection with the computations made for steady-state conditions. The model computes steady-state pressure, soil moisture and N-species concentration profiles. STUMOD-FL could also be applied to any chemical that attenuates through sorption and reaction although this function has not been incorporated into the model at this time. The rates of nitrification and denitrification reactions are highly dependent on a number of environmental factors including substrate concentration, temperature, redox conditions, and diffusion of reactants and products (the latter two are closely related to soil moisture). The effect of moisture content on nitrification and denitrification is calculated based on the soil moisture profile. The model also accounts for the effect of temperature. Reactions rates are dependent on the concentration of the contaminant described via Monod kinetics. The input parameters are operational parameters such as hydraulic loading rate, input effluent concentration and soil properties. The model can accept both ammonium and nitrate effluent concentrations. The output is expected performance represented by constituent concentrations and removal efficiency.

STUMOD-FL can handle up to 4 different soil layers each with different properties. Thus, soil hydraulic properties can be varied on layer by layer basis. The first layer at the infiltrative surface is a biomat and is assigned biomat hydraulic properties. This layer is typically in the range of 0.5 to 5 cm thick (USEPA, 2002a). The remaining layers are assigned native soil properties. Each of these layers can further be divided into several segments for computational purposes. Generally, nitrification and denitrification occur within certain soil moisture ranges (Bollman and Conrad, 1998; Schjonning et al., 2003). An analytical solution was applied to calculate the profile of suction head in each segment, starting from the bottom segment. The suction head is used to estimate moisture content. We use water content as a surrogate for the many parameters that control oxygen diffusion and uptake (Tucholke, 2007).

MATERIALS AND METHODS

STUMOD-FL is a practical tool that can provide cost-effective evaluations on nitrogen removal strategies in soil treatment units. The overall procedures used to calculate removal efficiency are shown in Figure 1. An analytical solution is used to calculate profile of pressure and moisture content based on Darcy’s law and the relationship between suction head and moisture content. The chemical transport component is based on simplification of the general advection dispersion equation

(ADE), which is based on mass balance theory. The model is relatively simple to use but detailed enough to account for important fate and transport processes such as advection, sorption, nitrification and denitrification. The model is parameterized for evaluation of nitrogen removal in soil treatment units. The input parameters for nitrogen transport and transformation for the biomat and the native soil layers were derived based on a thorough literature review and statistical analysis of the available data. . The model calculates change in moisture content with depth; thus, the effect of soil moisture on nitrification and denitrification rate can be determined. It also accounts for the effect of temperature, carbon content and of nitrate concentration on denitrification rates.

Relative to other existing analytical models, STUMOD-FL is more detailed with respect to the soil-hydraulics and treatment processes as it incorporates many of the transformation processes that are usually implemented in complex models. Thus, STUMOD-FL requires input data influencing the hydraulics and transformation and transport of nitrogen for each soil layer (a biomat and the native soil layer below the biomat) (Figure 2). Default input parameters for hydraulics, nitrification, sorption and denitrification are provided. A graphical user interface allows users to choose a soil type upon which the model will automatically populate the default input parameters. Users can also populate the model with their own site specific data if available. Model estimates of treatment performance with depth are then obtained based on the default values or user specified inputs. For STUMOD-FL, relevant soil parameters including saturated hydraulic conductivity, residual water content (θ_r), water content at saturation (θ_s), and the van Genuchten fitting parameters α and n were estimated based on an assessment of soil data records from the Florida Soil Characterization Data Retrieval System (University of Florida, 2007). Model output includes soil moisture content, nitrogen species concentrations (Figure 3) and the fraction of total nitrogen reaching the aquifer, and mass flux.

Root water and nutrient uptake modules are included in STUMOD-FL. Various approaches have been used to calculate root water uptake. Some approaches use crop coefficients (K_c) with potential evapotranspiration to estimate specific crop evapotranspiration rates and other methods use soil water suction. Because STUMOD-FL calculates the suction profile, we chose a more rigorous approach where the root water uptake is a function of the soil water pressure head, root characteristics, and potential evaporative demand. We use coefficients for the effect of soil moisture status using a soil moisture profile calculated by STUMOD-FL and root distribution to calculate the actual evapotranspiration (ET) from potential evapotranspiration (ET_0). Hargreaves Method is used to calculate ET_0. The Hargreaves-1985 equation is one of the simplest and most accurate empirical equations used to estimate ET_0 (Jensen et al., 1997). The nutrient uptake is calculated as sum of a passive and an active uptake. The passive uptake represents flow of nutrients into roots associated with upward flow of water supplying the plant transpiration demand. The passive uptake is driven by ET and controlled by crop nutrient demand and nutrient availability. It is computed as the product of nutrient concentration and upward water flux (ET). When nutrient availability is not limiting, passive uptake is determined by nutrient demand and ET. When nutrient availability is limiting, the passive uptake is reduced to a value less than the crop demand. The nutrient availability is dependent on the soil water nutrient concentration calculated by the model and changes with depth due to the nitrification and denitrification process. Thus, the passive nutrient uptake varies with depth as concentration changes. The passive uptake is the primary uptake mechanism in STUMOD-FL; however, an active uptake component representing the movement of nutrients into the roots induced by other mechanisms than mass flow is included in STUMOD-FL. An active uptake compensates for crop demand not met via passive uptake but the compensation is also limited by the concentration of nutrients. In addition to the nutrient availability, both passive and active nutrient uptake are affected by root depth and distribution.

For OWS, knowledge of groundwater fluctuations is beneficial because the amount of nitrogen reaching the water table is affected by the separation distance between the water table and the soil treatment unit. Two options are provided for the location of the water table. Users can enter either a known water table depth or use the model calculated water table as determined by a water table fluctuation model included in STUMOD-FL. Various approaches have been used to assess the seasonal fluctuation of a water table and several different analytical models have been developed. In STUMOD-FL, a water table fluctuation model developed by Park and Parker (2008) was adapted to Florida aquifer conditions. The model calculates water table fluctuations based on discrete records of precipitation such as daily or monthly precipitation data. It requires precipitation time series and other inputs related to soil properties that control the reduction in ground water level with time when precipitation is not occurring and water table build up during precipitation. OWS could then be designed on a more conservative approach based on a maximum precipitation year or a year with high precipitation to evapotranspiration ratio. The model was selected because it is a physically based model and was specially developed for aquifer response to precipitation time-series.

RESULTS and DISCUSSION

Nitrogen is applied to the soil treatment unit mainly as ammonium nitrogen (NH4-N). NH4-N is converted to nitrate nitrogen (NO3-N) through the nitrification process. NO3-N concentration starts increasing in the shallow depth as NH4-N is converted to NO3-N and starts to decline there after due to the denitrification process into gaseous nitrogen (Figure 3). The rate of this change depends on various factors discussed earlier including the water content and temperature. STUMOD-FL captured the general trend in the field with respect to soil type and hydraulic loading rates. For most soil types STUMOD predicted ammonium conversion to nitrate within the first foot below the trench infiltrative surface. Ammonium persisted relatively deeper below the trench in finer grained soils and high loading rates due to low nitrification rates caused by high predicted water content.

CONCLUSION

STUMOD-FL is based on principles of water movement and contaminant transport using an analytical solution to calculate pressure and moisture content profiles in the vadose zone and a simplification of the general advection dispersion equation.

STUMOD-FL has been adapted to Florida-specific soil and climate data that includes parameters representing dominant soil properties, account for ET, account for the effect of high/seasonally variable water tables on nitrogen removal in soil. STUMOD-FL primarily addresses the most common operating conditions associated with OWS. The current version includes an aquifer module for ground water transport. The integration allows estimation of nitrogen concentration reaching the water table and at well or surface water location down gradient a source. Default soil properties determined based on median values obtained from the Florida Soil Characterization Data Retrieval System are included in the model. However, STUMOD-FL allows user-specified input and can be calibrated to site-specific data. The output is the expected steady-state performance (i.e., constituent concentration). Model outputs provide insight into the behavior of soil treatment and quantitative estimations of nitrogen removal as affected by a range of conditions. STUMOD-FL includes the effect of plant nutrient uptake. Both nitrate (NO_3) and ammonium (NH_4) species are assumed to be equally available to plants. STUMOD-FL estimates the water table fluctuations in response to precipitation based on the equation for one-dimensional flow in unconfined aquifers.

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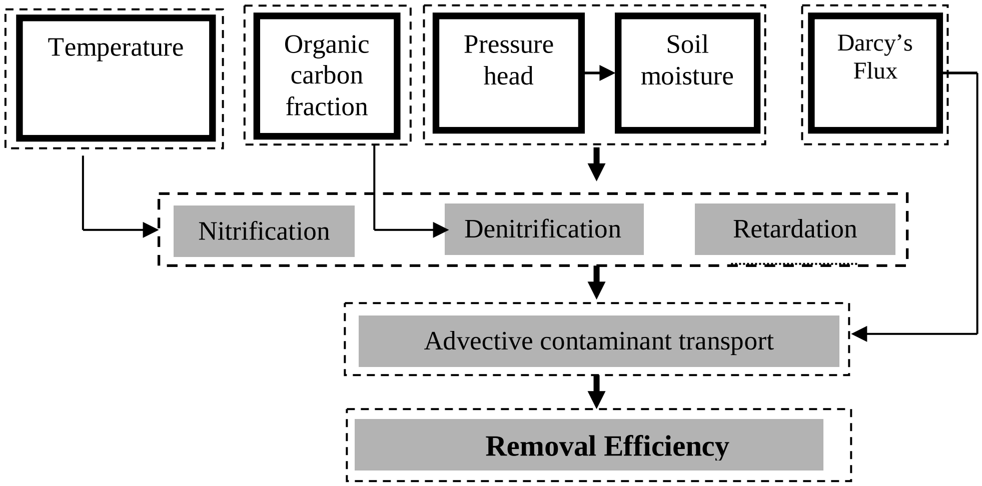

Figure 1. Flow chart of procedures in STUMOD

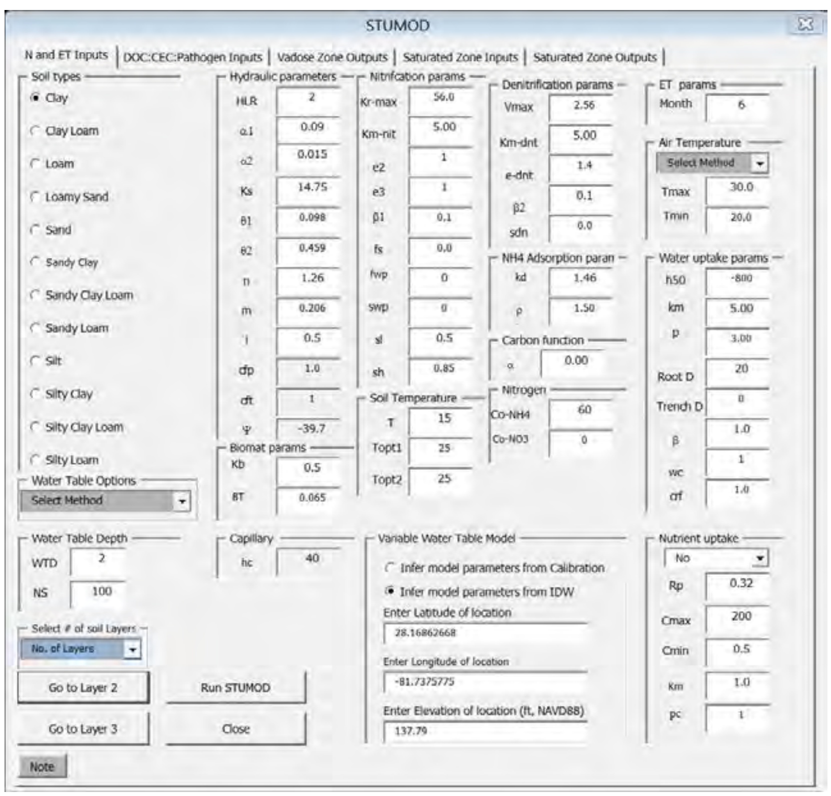

Figure 2. STUMOD-FL graphical user interface.

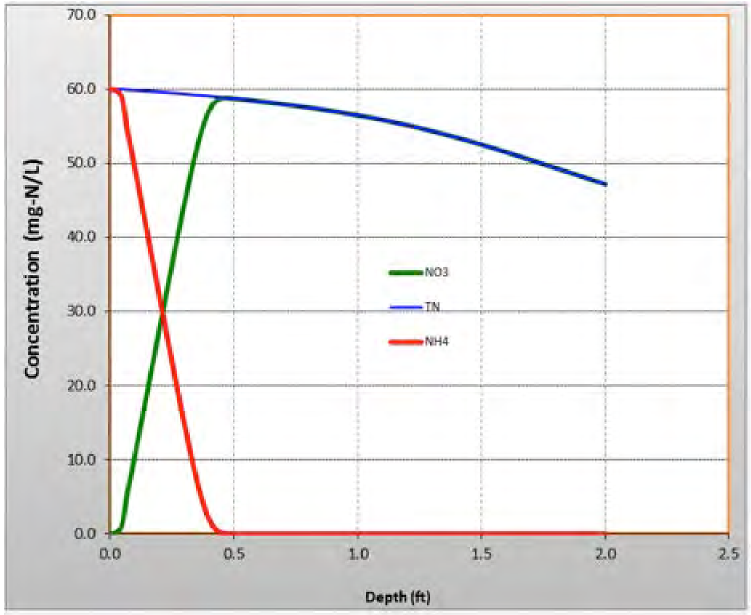

Figure 3. Illustration of STUMOD-FL output representing the concentration of nitrogen below the center of a soil treatment unit.

Development of an Analytical Groundwater Model for Fate and Transport of Nitrogen from Onsite Wastewater Systems

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ABSTRACT

Understanding the fate and transport of wastewater constituents in groundwater is an important aspect in the design and management of onsite wastewater systems (OWS) as well as maintaining groundwater quality. Numerous mathematical models exist for evaluating fate and transport however the robust numerical models that exist are complex and difficult to use. Screening level models, while they are easier to use, may not accurately predict fate and transport from OWS. As part of the Florida Onsite Sewage Nitrogen Reduction Strategies (FOSNRS) project, both vadose zone (STUMOD-FL) and shallow groundwater models are being developed to evaluate the impacts of OWS on water quality. The groundwater model is discussed in this paper, and improves on previous screening level models by accurately considering the geometry of the OWS and by avoiding the mathematical assumptions that compromise the results of other screening level models. Comparison of results from the groundwater model with robust numerical models indicates it accurately predicts fate and transport. Algorithms have been added to give users additional methods to visualize the contaminant plume, calculate mass flux and estimate model parameters. Coupled with STUMOD-FL, the groundwater model gives users an improved tool to evaluate the impact of design and management of OWS on groundwater quality.

Evaluating the impact of OWS on groundwater quality is an important aspect of design and management. The primary constituents of concern in household wastewater are nutrients, pathogens and organic compounds derived from pharmaceuticals. Nutrients, such as nitrogen and phosphorous, are by far the most prevalent constituents in household wastewater (Crites and Technobanoglous, 1998, EPA, 2002, Lowe, 2006, McCray, 2009, McCray, et al., 2005). Nitrogen is highly mobile in groundwater because it reaches groundwater primarily in the form of nitrate after undergoing nitrification in the vadose zone (Fischer, 1999). Once nitrate has reached groundwater it can easily move offsite. Nitrate has been linked to health problems in humans and is known to degrade surface water quality in nitrogen limited aquatic systems. Because of its detrimental effect, the US EPA has established a maximum contaminant level (MCL) of 10 mg/l nitrate nitrogen in drinking water (EPA, 2009). Nitrogen levels well below 1 mg/L have been established for ecosystem protection under water body specific TMDLs in some locations.

The methods that are implemented to evaluate the impact of OWS on groundwater are generally arranged in order of increasing complexity (McCray, 2009). This approach minimizes costs to the user while protecting groundwater and surface water resources. STUMOD-FL has been designed to evaluate the fate and transport of wastewater constituents in the vadose zone while the new groundwater model will provide an understanding of fate and transport in groundwater. The groundwater model improves upon previous screening level models by utilizing an improved solution to the advection dispersion equation while minimizing input requirements.

MATERIALS AND METHODS

The groundwater model is based on an analytical solution to the advection dispersion equation derived by Galya (1987). This solution considers a horizontal plane source similar in geometry to the soil treatment unit of an OWS. This is an improvement on many of the previous screening level models that exist because many of these models utilize a solution that considers a vertical plane source. The Domenico and Robbins (1985) solution considers a vertical plane source and is widely utilized because a close form solution exits. Assumptions must be made concerning the mixing layer beneath the OWS and the dimensions of the vertical plane source (Figure 1). Determining the dimensions of the effective vertical plane source in the Domenico and Robbins (1985) solution is not a straight forward process and can impact model predictions. Incorrect dimensions for the source plane or erroneous source concentrations may result in over or under predictions of nitrate concentrations and loads. These assumptions of a mixing layer and source concentration need not be made for the solution implemented in this groundwater model, which utilizes a horizontal plane source. The horizontal plane source dimensions are effectively the footprint of the OWS which are easily measured in the field or obtained from design specifications (Figure 2). The source concentration at the water table is evaluated using STUMOD-FL.

The horizontal plane source solution is also an improvement over previous screening level models because it does not make the same mathematical assumptions that Domenico and Robbins (1985) employed. In order to derive a closed form solution to the advection dispersion equation, Domenico and Robbins assumed that the time term in the transverse (horizontal and vertical) spreading terms could be approximated as travel distance divided by the seepage velocity. Srinivasan et al. (2007) show that this assumption causes the Domenico and Robbins solution to under predict concentrations when the longitudinal dispersivity is not equal to zero. Under predicting concentrations is undesirable in screening level models because it can lead to inappropriate design and management of OWS. The horizontal plane source solution does not make assumptions about the transverse spreading time which makes it mathematically correct. Because it does not make this assumption, the horizontal source plane is not a closed form solution and requires numerical integration. Two numerical integration techniques were evaluated to ensure there was no loss in accuracy due to the use of numerical methods. Solutions using the trapezoidal rule were compared against a robust adaptive quadrature method that refines the discretization along portions of the function that change rapidly. No differences were observed between the outputs using the two techniques.

RESULTS AND DISCUSSION

As part of the validation process outputs from the horizontal plane source model were compared to a steady state numerical model simulation using MODFLOW and MT3D to simulate the transport of a conservative contaminant. The numerical model was constructed using 161 columns, 624 rows and 21 layers for a total of 2.1 million cells. The spatial discretization of the rows, columns and layers along the centerline was designed to eliminate the influence of numerical dispersion. It is well known that grid peclet numbers less than or equal to 0.25 are necessary to avoid numerical dispersion (Zheng and Bennett, 2002). The large number of cells was necessary to simulate the equivalent of what is considered by the horizontal plane source solution while maintaining a fine spatial discretization. The numerical model required 24 hours to execute on a computer with an Intel Xenon, 4 core processor. This illustrates the advantage of using analytical solutions which take seconds to execute on computers with standard hardware. Results were compared at depths from 0 – 9 m below the water table along 735 m section of the steady state plume centerline. The largest root mean squared error observed between MODFLOW\MT3D and the horizontal plane source did not exceed 0.01 mg/l. This indicates that the horizontal plane source solution accurately predicts contaminant transport under conditions for which it was derived, one dimensional flow and three dimensional dispersion.

As with any analytical solution to the advection dispersion equation the horizontal plane source makes assumptions to arrive at the analytical solution. The primary assumption made by the horizontal plane source solution is one dimensional flow. One dimensional flow is an additional assumption made by the Domenico and Robbins solution. In an effort to evaluate the effect of this assumption on the horizontal plane source outputs an additional numerical model was constructed. The same spatial discretization and transport parameters were used as in the first test case. The primary modification that was made was a heterogeneous hydraulic conductivity field. The heterogeneous hydraulic conductivity field was constructed using a random number generator. The random hydraulic conductivity values varied over two orders of magnitude, which is representative of what has been measured at the FOSNRS field site. The average root mean squared error for the heterogeneous hydraulic conductivity field was slightly higher than the homogenous case though the highest observed error did not exceed 0.01 mg/l. These results are encouraging though it is important to point out that the effect of geologic structure was not considered in this test case.

Similar to STUMOD-FL, which was designed to be user friendly, the groundwater model was designed to minimize input data requirements. The general implementation of the horizontal plane source requires the user to input transport parameters as well as parameters for the numerical integration. Parameters for numerical integration have been fixed in the groundwater model to provide the user steady state results for a wide variety of conditions. The stability of numerical integration was evaluated to ensure that users would be provided accurate results under a wide range of conditions. The user must specify the dimensions of the OWS footprint and the water table gradient. As previously mentioned the OWS footprint can be easily measured in the field or obtained from design specifications. The hydraulic gradient can be obtained from two to three piezometers or in event that no wells exist in the vicinity of the OWS, it can be approximated as the gradient of the land surface. If latitude and longitude coordinate data are available for three points the groundwater model has an additional algorithm that can calculate the hydraulic gradient and direction of groundwater flow relative to north. Additional parameters are obtained automatically from STUMOD-FL simulations though the user may still modify the parameter values if they wish. Dispersivity values must be input by the user, alternatively the user may select to automatically estimate these values using the method of Xu and Eckstein (1995). Dispersivity values calculated using this method have been compared to several tracer test studies conducted on surficial sand aquifers and have been found to be the most representative of observed values (Bitsch and Jensen, 1990, Mallants, et al., 2000, Sudicky, et al., 1983).

Additional algorithms have been added to groundwater model that provide the user with a mass flux estimate in addition to concentration estimates provided by the original formulation. Mass flux is calculated at a vertical plane at a user specified point down gradient of the source. Such a point could represent a surface water body where the user wishes to evaluate the impact of OWS. In addition to mass flux, algorithms have been added that provide visualization in the vertical and horizontal planes, effectively providing three dimensional visualization. Further development is ongoing to provide the user a method to simulate the effect from multiple spatial inputs at locations down gradient.

CONCLUSIONS

The groundwater model that has been developed for the FOSNRS project for linkage to STUMOD-FL, provides a screening level model to evaluate the effects of OWS on groundwater quality. It is an improvement over previous screening level models because it accurately captures the geometry of the OWS and does not require any explicit assumptions concerning a mixing layer or source concentration. It is mathematically correct and does not suffer from the mathematical errors encountered when using the Domenico and Robbins solution. The numerical evaluation of the horizontal plane source solution does not compromise the accuracy of the solution. Corroboration of the analytical solution utilized in the groundwater model against a numerical model shows that the analytical solution is capable of accurately predicting contaminant transport. The implementation of this analytical solution in the groundwater model has been designed to minimize input requirements from the user thereby making it user friendly. Inputs that are required are easily obtained by the user making this a powerful tool for evaluating the impacts of OWS on groundwater quality and ensuring sustainable management.

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Figure 1. The necessary assumptions about a mixing layer and source plane concentration are illustrated in the figure below(Guyonnet, 2001)

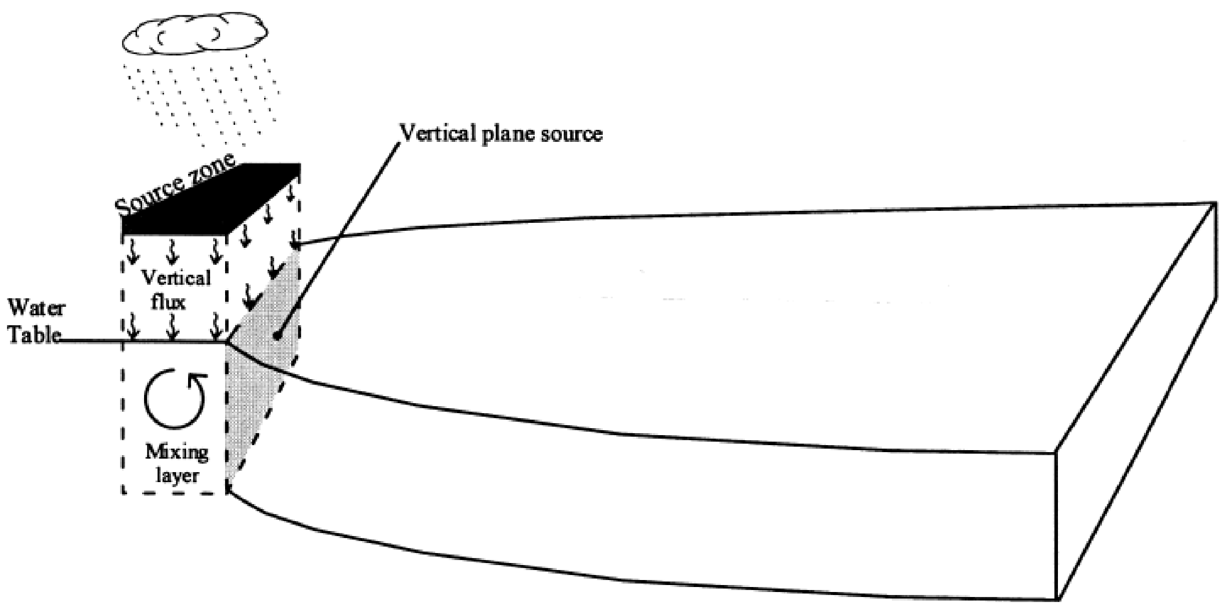
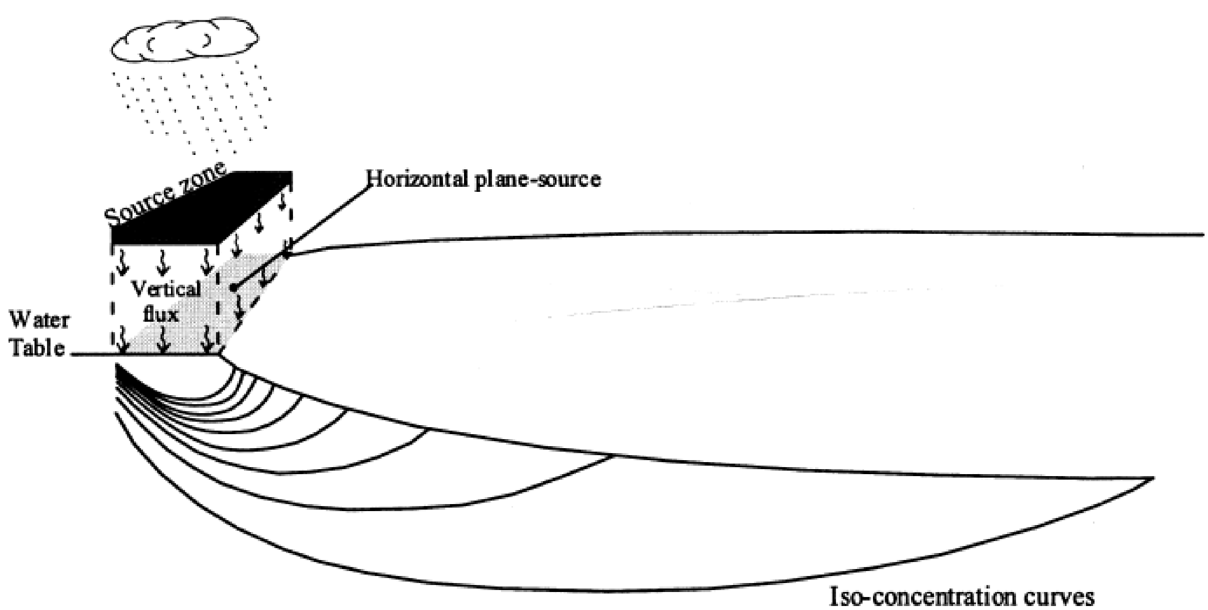


Figure 2. The horizontal plane source solution to the advection dispersion equation does not require any assumptions concerting a mixing layer or source plane concentration as it is directly calculated by the vadose zone model (Guyonnet, 2001)



Understanding and Interpreting Oxyaquic Conditions

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ABSTRACT

Hydropedologic studies related to seasonal saturation and hydraulic conductivity add to our knowledge to make accurate land use interpretations, particularly as related to land application of waste (liquid and solids) and many urban land uses. Soils mapped in the Carolina Slate Belt in the southeastern region of the United States, including the benchmark Tatum and Chewacla Series, are no exception to this and proper identification of seasonal saturation in these soils is critical as urban and suburban development increases in this region. Soils related to the Catena may lack the typical 2 chroma redox depletions commonly used to identify seasonal saturation even though high water table is often directly observed in these soils. When a seasonal high water table is determined, the soil may be classified as oxyaquic. However, if 2 chroma depletions are absent (or present at deeper depths than seasonal saturation) local or state land use codes may misidentify the depth to saturation. Therefore, even when a soil is classified as oxyaquic, local and state codes do not always interpret this as a limitation for waste treatment and dispersal purposes. The result is that soils in this region's toposequence (particularly Georgeville-Tatum-Lignum-Chewacla) may be inappropriately used for waste disposal and other purposes. Along with proper identification of saturation, soil hydraulic conductivity measurements are needed. The soils listed above all have similar hydraulic conductivities listed in their current interpretations, yet anecdotal field data from consultants indicate a wide range in measured values. The hydropedology data from this study has shown that the redox depletions in this area are indeed related to saturation. This fact has been debated by consultants and local health departments. Prior to this study one prevailing view was that the low chroma features were simply due to stripping or leaching of Fe in old cotton or tobacco fields and in no way was related to saturation. Based on the evidence in this study the interpretation of the redox depletions, oxyaquic conditions, and occurrence of episaturation will need to be reconsidered. In general in situ measurements are lower and have a lower standard deviation as compared to laboratory measurements. However, this difference is not statistically significant. The measured field values also fall well below the NRCS hydraulic conductivity range of 1.5 – 5.1 cm/hr (0.6 - 2.0 in/hr) per the Orange County Soil Survey for a Georgeville, Lignum, Taurus and Chewacla soil series.

INTRODUCTION

Saturation has long been related to the presence of iron based redoximorphic features (Daniels et al., 1971; Bouma, 1983; Franzmeier et al., 1983; Pickering and Veneman, 1984; Evans and Franzmeier 1984; Jacobs et al. 1993; Megonigal et al., 1993, Veneman et al., 1998; West et al., 1998; Hayes et al., 2000; He et al., 2002, 2003; Morgan and Stolt, 2006; Severson et al., 2008). Depth to gray, low (≤ 2) chroma redox depletions is traditionally used to determine the depth to a seasonal high water table (SHWT) in order to assign drainage classes to soils (Evans and Franzmeier, 1986; Schoeneberger et al., 2001; Soil Survey Staff, 1999; Vepraskas, 1999) or for other land uses (Lindbo et al., 2004; NCAC, 2005).

Formation of iron depletions requires stagnant, oxygen-free water, a viable source of organic matter, temperatures above 5° C, a source of iron, and a sufficient amount of time (Vepraskas and Faulkner, 2000). It was found that the average duration of saturation to create iron reducing conditions in the Coastal Plain region of North Carolina is 21 days or greater for depths between 0 and 60 cm (He et al., 2003; He, 2000). The amount of saturation, reduction and redox feature abundance usually increase with depth (Vepraskas, 1999), yet the exact duration and frequency of saturation and reduction at the depth of the first occurrence of redox depletions is often unknown without the aid of monitoring. He's work (He et al., 2002; 2003; He, 2000) showed that the percentage of gray colors (chroma ≤ 2) at a given depth could be correlated to the duration and frequency of saturation.

Most often Fe-depletions of chroma 2 or less are the features used to determine seasonal water table depth. Other iron-based redox features, such as depletions with chroma ≤ 3 and iron concentrations (≥ 6 chroma), are related to water-table fluctuation but generally reflect saturation for shorter periods of time than ≤ 2 chroma depletions (Severson et al., 2008; Genther et al., 1998; West, et al., 1998; Evans and Franzmeier, 1986; Vepraskas and Wilding, 1983). These other features that are now being correlated to saturation frequency or duration, used in conjunction with chroma 2 depletions, tend to lead to more accurate assessment of a SHWT than the use of ≤ 2 chroma redox depletions alone (Morgan and Stolt, 2006; Veneman, et al., 1998).

Redoximorphic features (RMFs) that result from reduction and oxidation cycles occurring over many years represent long-term indicators of seasonal groundwater elevation. However, drainage ditches and subsurface tile drains alter hydrology, resulting in the redoximorphic features being out of equilibrium with the hydrology (Lindbo, 1997; Hayes and Vepraskas, 2000). Features observed at hydrologically altered sites are referred to as relict features (James and Fenton, 1993). Some of these relict features are distinguished in the field by having sharp boundaries with the matrix, while contemporary redox features usually have diffuse boundaries and are indicative of a fluctuating water table (Vepraskas, 1999).

Knowledge of morphologic and saturation relations are important for forming the perspective of an urban land use such as wastewater treatment and dispersal. Proper wastewater treatment depends on aerobic soils; therefore, an accurate identification of the depth to seasonal saturation (frequency and duration) is a critical siting parameter. Understanding this relationship of frequency and duration of saturation to morphology will aids land managers and environmental health officials with proper classification of the soils and in protecting surface and groundwater quality.

The Soil Science Society of America (https://www.soils.org/publications/soils-glossary) defines soil permeability as “the ease with which gases, liquids, or other substances can flow through it.” Soil permeability, also known as intrinsic permeability (identified by k, with units of length squared, as m^2), depends only on the soil (or other porous media), but is independent of the properties of the liquid flowing through it. Saturated hydraulic conductivity, defined as a measure of the ability of soil to transmit water, on the other hand, depends on both the soil and water properties. Saturated hydraulic conductivity (generally identified by K_{sat} or K_s) in the vadose zone of a soil can be used as an indication of the ability of that soil to transmit water. In conjunction with other soil properties (e.g., texture, clay mineralogy), K_{sat} can provide valuable information regarding soil wetness and various processes associated with soil water content.

A number of field and laboratory procedures are available for determining the K_{sat} of the soil. One of the most convenient methods for in situ measurement of K_{sat} of the unsaturated (vadose) zone is the constant-head well permeameter technique (also known as shallow well pump-in technique and bore hole permeameter method) (Amoozegar and Warrick, 1986; Amoozegar and Wilson, 1999). Briefly, to measure K_{sat} by this procedure a cylindrical auger hole is dug to the desired depth. Water is then applied to the bottom of the hole under a constant head and allowed to infiltrate the soil. After reaching steady-state, the rate of water flow into the soil is determined. Saturated hydraulic conductivity is then calculated using the steady-state rate of water flow into the soil (Q), the constant depth of water in the hole (H), and the radius of the bottom part of the cylindrical auger hole (r). The Glover model (Zangar, 1953)

$$K_{sat} = [\sinh^{-1}(H/r) - (1 + r^2/H^2)^{0.5} + r/H]Q/(2\pi H^2)$$

is recommended for calculating K_{sat}. Other models and approaches are also available for determining K_{sat}. The Glover model, however, is analytical and rather simple; it does not result in any unacceptable negative values, and does not require estimation of any soil parameter (Amoozegar, 1989b).

Different procedures and devices area available for maintaining a constant depth of water at the bottom of an auger hole and measuring the rate of water flow into the soil. The Compact Constant Head Permeameter (CCHP) is perhaps the most convenient device that allows measurement of K_{sat} from near the soil surface to 4 m depth using a small diameter (4- to 8-cm diameter) auger hole (Amoozegar, 1989a, 2004). The auger hole can be dug to the desired depth by a hand auger. Alternatively, a mechanical auger or probe can be used to bore a hole to within 30-cm of the depth of measurement. The bottom of the hole, however, must be dug by a hand auger and be cleaned with a planer auger to obtain a cylindrical hole. The diameter of the bottom portion of the hole (i.e., where a constant depth of water is maintained), and the constant depth of water in the hole must be determined accurately.

Although 2 chroma redox depletions have long been used to determine seasonal saturation depth, the soils in the Carolina Slate Belt may lack the 2 chroma depletions while 3 and 4 chroma depletions are common. The question then arises as to how to interpret this morphology. Assuming the 3 and 4 chroma depletion form the same way as 2 chroma (Vepraskas, 1999), can these features be equated to the same saturation as 2 chroma or do they represent less frequent or shorter duration events (Severson et al., 2008)? Furthermore, oxyaquic conditions are often used when describing soils in this toposequence. These conditions will be either confirmed or refuted with data (saturation and Eh) from this study.

“Tiger dirt” is a regional or colloquial term used by soil consultants and environmental health specialists to describe common color patterns in Slate Belt and related soils. The pattern consists of vertical and horizontal (along root channel and structural pores) streaks of lower chroma (3 and 4 chroma) material as compared to the matrix (usually 6 and 8 chroma). There exists a difference of opinion as to how to interpret this pattern. Often consultants interpret these as old root channels where preferential flow and leaching of iron has occurred. They often suggest that since these are observed on eroded agricultural fields that farming practices are the cause of the features. On the other hand, regulators interpret these features as indicating saturation when they have a low enough chroma (2 chroma) or at least suggesting limitation due to wetness. This study will be conducted in areas where “Tiger Dirt” is observed within the toposequence, thus the hypothesis that this pattern is due to saturation can be tested.

In-situ hydraulic conductivity measurements are often used in land use evaluation in this region. The values may vary by several orders of magnitude at a given site. It is necessary to determine the source of at least some of this variability in order to be able to best interpret the results. Since hydraulic conductivity is dependent on soil properties as well as methodology this study will relate morphology to hydraulic conductivity measurements. It is likely that structure and consistence will be strongly related to the hydraulic conductivity. One question that needs to be answered is whether or not in-situ conductivity measurements are more appropriate than laboratory techniques using intact or repacked soil samples. This study will compare the laboratory and in situ procedures for the soils investigated.

METHEDS AND MATERIALS

The project area is in the thermic, Carolina Slate Belt Sol System and is located approximately 8 miles north of Hillsborough in Orange County, NC. The site, Breeze Farm, is owned by North Carolina State University (Figure 1 and 2). It consists of approximately 250 acres of hay, pasture, row crop and forested land. The site contains uplands to flood plains adjacent to several creeks that run through the property. Based on the soil survey and Orange County GIS the soils on the site include; Georgeville, Tatum, Lignum, Cid, and Chewacla. Preliminary investigation on the site have confirmed these series as well as unclassified soils with a moderately well and somewhat poorly drained drainage classes. In addition, several areas on the site have been observed where oxyaquic conditions are likely to occur. Transects are located to cross from the well-drained Georgeville and Tatum through the less well drained soils and terminate in the flood plain where Chewacla soils occur (Figure 2).

An additional site is located in Alamance County is in the thermic, Carolina Slate Belt Soil System and is located approximately 20 miles west of Hillsborough in Orange County, NC (Figure 1 and 2). The site, Sykes Farm< is owned by the North Carolina Wildlife Commission. It consists of over 200 acres of recently clear cut (2009) forest land. After clear cutting the site has been allowed to return to natural vegetation. The site consists of several broad ridges that are dissected by ephemeral and intermittent streams. Based on the soil survey the soils on the site include; Efland, Georgeville, Tirzah, Wahadkee, Chewacla. Transect is located to cross from the well-drained Georgeville through the less well drained soils and terminate in the flood plain where Chewacla soils occur (Figure 2).

Three transects are located in the forest or inactive pasture/forest edge. Specific site locations for wells and redox probes (as applicable) were determined through reconnaissance boring. Observation wells and redox probes are installed on each transect following standard procedures. Wells extend to approximately 80 cm, to the depth of seasonal low water table, or to a lithic contact whichever is shallower. Redox probes are installed at depths similar to the piezometers. A recording rain gauge is installed on each site.

Observation pits on the Breeze Farm site were dug at least 2 meters away from the transects in order to avoid disturbing the instrumentation. Soil profile descriptions were made by MLRA Soil Scientists. Samples were taken from each horizon for analysis by the USDA, NRCS-NCSS laboratory.

Soil cores will be taken at several depths for laboratory determination of hydraulic conductivity (see subsequent section) on one transect at Breeze Farm. In-situ hydraulic conductivity will be measured in triplicate in at similar depths. Procedures and data analysis will follow the standard methods (Amoozegar, 2004; Amoozegar and Wilson, 1999).

For laboratory measurement of saturated hydraulic conductivity a minimum of 30 intact soil cores were collected from three different horizons/depths (10 samples each) of the soils on one transect. The horizon/depth of sampling matches the depth and locations where in situ K_{sat} measurements were conducted. The samples were collected by either an Uhland type sampler (using a 6.5 to 7.5 cm diameter and 7.5 cm long cylinder). Hydraulic conductivity was measured by the constant head method (Amoozegar and Wilson, 1999). Falling head method was used for samples with very low hydraulic conductivity. Prior to conductivity measurements, all samples were saturated from the bottom by placing them in water in a plastic tub and raising the level of water in the tub within a 24- to 48-hr period. After complete saturation, the cores were placed in special funnels on a rack. Water was applied to the top of the cores and a constant head of water will be kept on top of the cores using a Mariotte bottle system (for a set up see Amoozegar and Wilson, 1999). The flow rate through the column was measured with time and a K_{sat} value will be calculated by Darcy's law for each measurement.

Results and Discussion

Only selected morphology (Table 1, Figure 3-7), hydrographs (Figures 8-10), redox data Figures 8-10), and hydraulic conductivity data (Table 2)for Transect 1 at Breeze farm are reported in this text. Full data set can be obtained by contacting the first author.

Figure 3 shows the uppermost Georgeville profile on Transect 1. Of note in this profile is the prismatic structure parting to angular blocky structure in the upper Bt. No wetness related redoximorphic features were observed although high chroma concentrations were described. These may be related to mineral weathering or clay accumulations. Roots were common and followed structural units.

Figure 4 is from the shoulder slope Georgeville profile. Unfortunately it was located approximately 45cm higher than the well and redox probe nest due to large trees in the way. Redox features are common starting at 12 cm with concentrations. At 84 cm redox depletions occur in channels and associated with dead roots. As with T1L1 Georgeville, the structure in the upper profile is prismatic parting to angular blocky structure. The low chroma depletions suggest this profile is saturated at 84cm long enough to form depletions. This would change the classification to Oxyaquic Kanhapludult.

The Lignum profile (Figure 5) was described and sampled approximately 30 cm topographically lower than the instrumentation nest due to topography and trees. It lacks the prismatic structure in the upper 2 profiles and has redox depletions occurring higher in the profile at 38 cm. The depletions occur in channels in the upper part of the profile and become more common with depth. In the BCt the depletions occur both horizontally as well as vertically forming what is locally referred to as "Tiger Dirt". The occurrence of deletions at 38 cm suggests this pedon be classified as Aquic Kanhapludult.

The next pedon moving down slope (Figure 6) was also sampled as a Lignum. The photograph suggests redox depletions occur as high as 47cm rather than the 67 cm reported in the profile description. This discrepancy is likely due to the photographed face and the sample face were not the same. Also of note in this pedon are the Mn concentrations starting at about 80 cm. Although redox depletions are common this pedon does not exhibit "Tiger Dirt" morphology.

The final pedon in this transect (Figure 7) was identified in the field as a Chewacla Series. Low chroma (<2) depletions did not occur until 66 cm yet 4 chroma depletions (around dead roots) are observed starting at 16 cm. Also of note in this profile are the Fe-Mn concentrations common at 148 cm.

Recording wells were installed at all locations with the exception of Breeze Farm T2L0. This site was not instrument due to proximity to the road and distance to the remainder of transect 2 (T2). Water levels were recorded twice daily and corrected for barometric pressure measured on site. Wells were installed to 250cm or auger refusal which ever was shallower. At selected locations a shallow well was installed between 90 and 100 cm. The locations for shallow wells were chosen based on landscape position and water table data from the deep wells. The water table depth (shallow (red line) and deep (blue line) as applicable), rainfall (purple lines), and depth to redox features (concentrations are the orange line and depletions are the green line) were plotted for each location (Figure 8-10).

A group of five platinum electrodes were installed at selected locations to measure Eh. At the pedons in the floodplain or toe slope position (T1L5, T2L4, T2L5, AL5 and AL6) the electrodes were installed at a depth of 30cm. At other locations the electrodes were installed at 100cm (T1L2, T1L3, T1L4, T2L2, T2L3, AL1 and AL4). Readings were approximately weekly during the high water table period of 2012 (January to April) and then once during the summer 2012 and fall 2012. The readings were correct to the standard hydrogen electrode by adding 250mV to the field reading. The data (high, average, low of the 5 readings) was plotted by Eh versus date for Breeze Farm T1L2 (Georgeville). Dashed line is theoretical line below which the system is Fe reduced. It is calculated by the formula: Eh = 595 – 60(pH).date and Eh (Figures 8-10).

Saturated hydraulic conductivity was measured at selected locations and depths at the Breeze Farm site (Table 2). Overall the laboratory measurements were 2 to 50 times higher than those made in the field. This is likely due to preferential flow within the soil and at the edges of the sampling tube. The in situ falling head measurements were comparable to those made with the compact constant head permeameter with the exception of the data for T1L5. It is possible that macropores accounted for the high K_{sat} in the falling head test. The K_{sat} values reported for these soils in the soil survey range from 1.5 – 5.1 cm/hr (0.6 - 2.0 in/hr). They are 2 to 100 times higher than what is reported in the field.

There is a drop in K_{sat} from the 45cm to 105cm depth in T1L2, L3, and L4. This suggests that the lower K_{sat} could cause the observed episaturation. The drop is not great but since saturation was observed in the shallow wells and redox concentrations were also within those depths it is likely that the lower K_{sat} is at least partials responsible. Furthermore there is an increase in K_{sat} with depth in the Georgeville T1L1 pedon. Neither saturation nor redox depletions were observed in that profile.

CONCLUSIONS

The hydropedology data from this study has shown that the redox depletions in this area are indeed related to saturation. This fact has been debated by consultants and local health departments. Prior to this study one prevailing view was that the low chroma features were simply due to stripping or leaching of Fe in old cotton or tobacco fields and in no way was related to saturation. Based on the evidence in this study the interpretation of the redox depletions, oxyaquic conditions, and occurrence of episaturation will need to be reconsidered.

In general in situ measurements are lower and have a lower standard deviation as compared to laboratory measurements. However, this difference is not statistically significant. The measured field values also fall well below the NRCS hydraulic conductivity range of 1.5 – 5.1 cm/hr (0.6 - 2.0 in/hr) per the Orange County Soil Survey for a Georgeville, Lignum, Taurus and Chewacla soil series.

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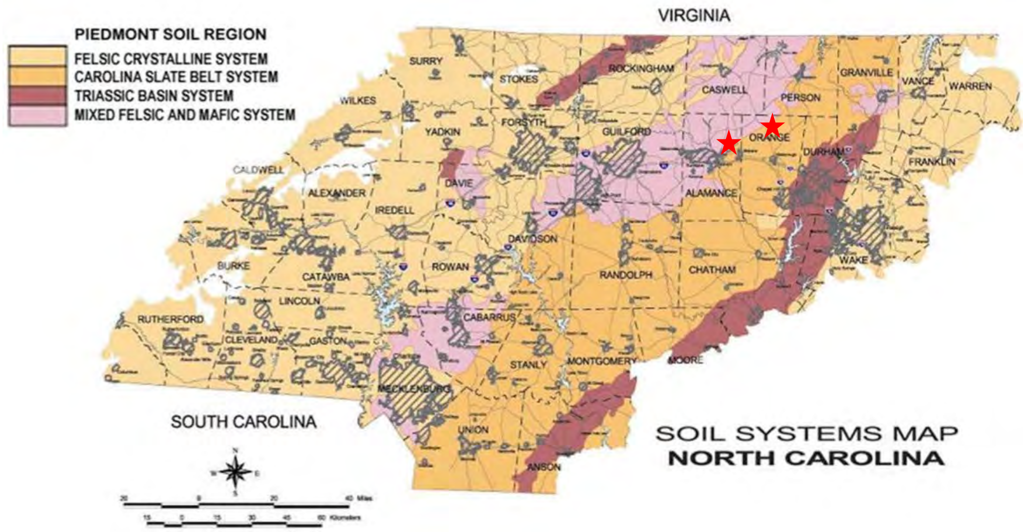

Figure 1: Breeze Farm site is in Orange County, Sykes Farm site is in Alamance County

Figure 2: Pedon locations for Breeze Farm site (left) and Sykes Farm site (right). Pedons are from well drained (0) to somewhat poorly drained (6).

Figure 3: T1L1 – Georgeville Profile.

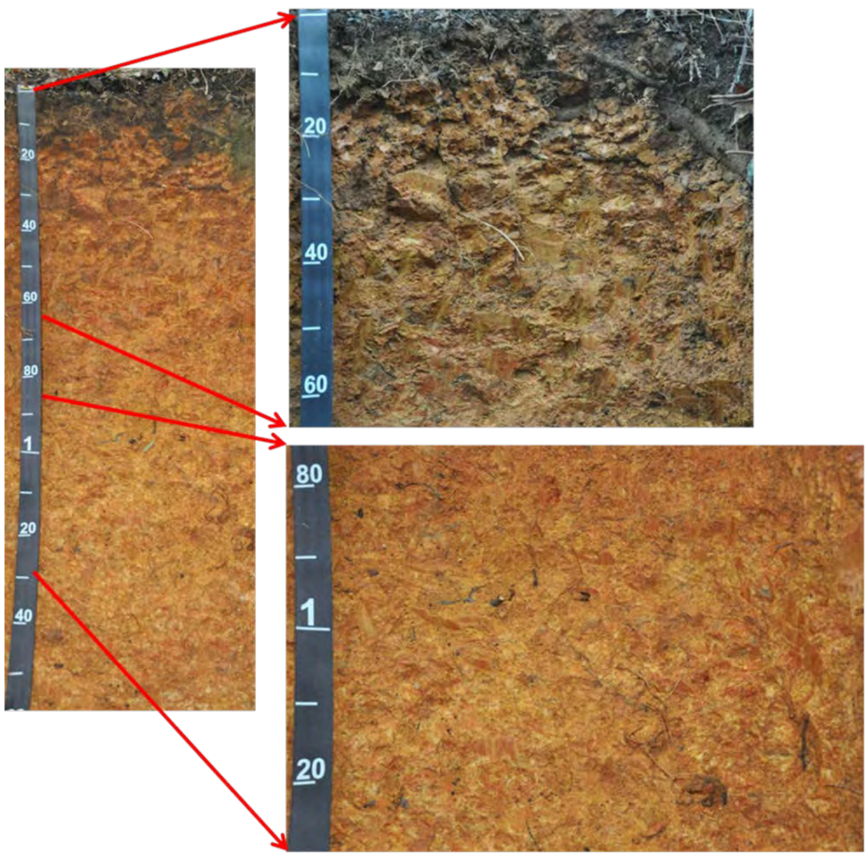

Figure 4: T1L2 – Georgeville Profile

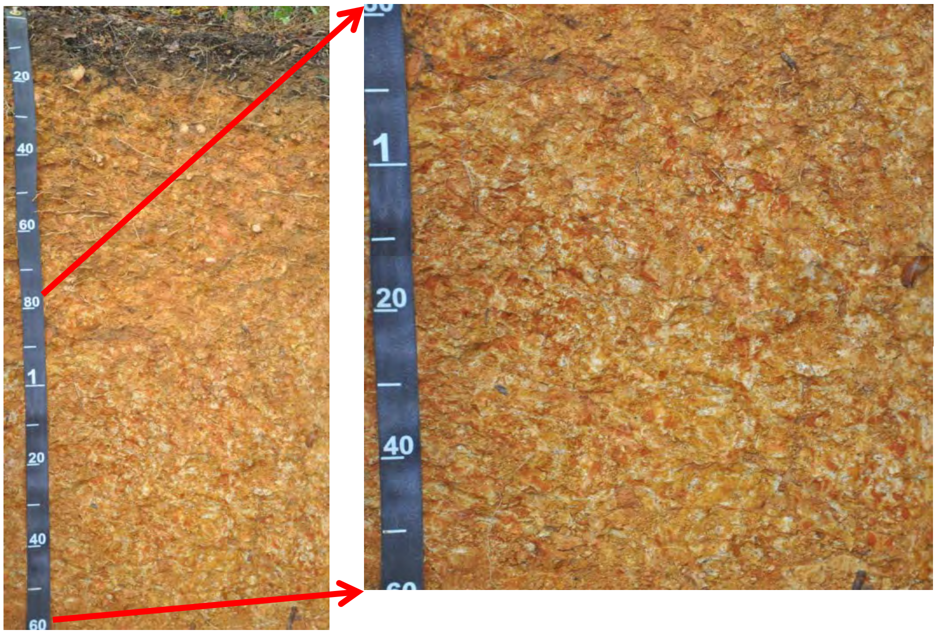

Figure 5: T1L3 – Lignum Profile.

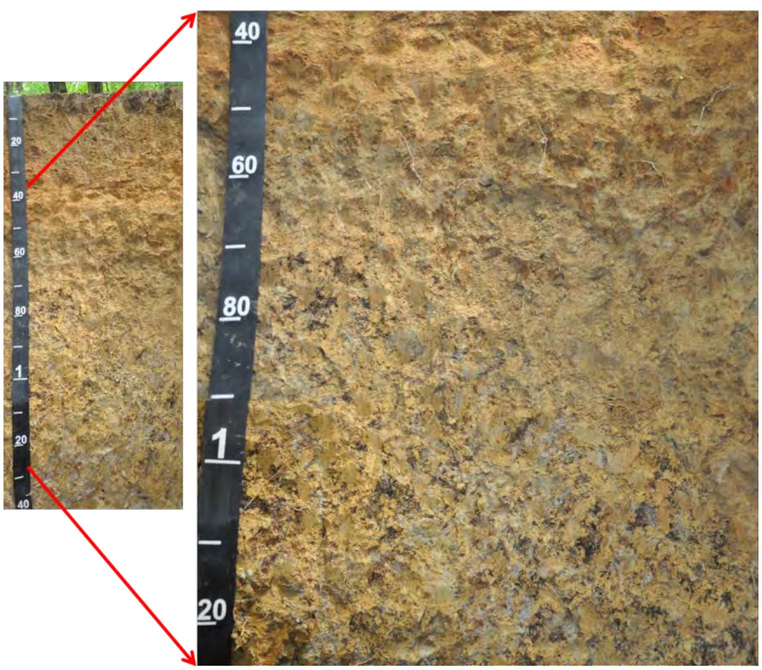

Figure 6: T1L4 – Lignum Profile.

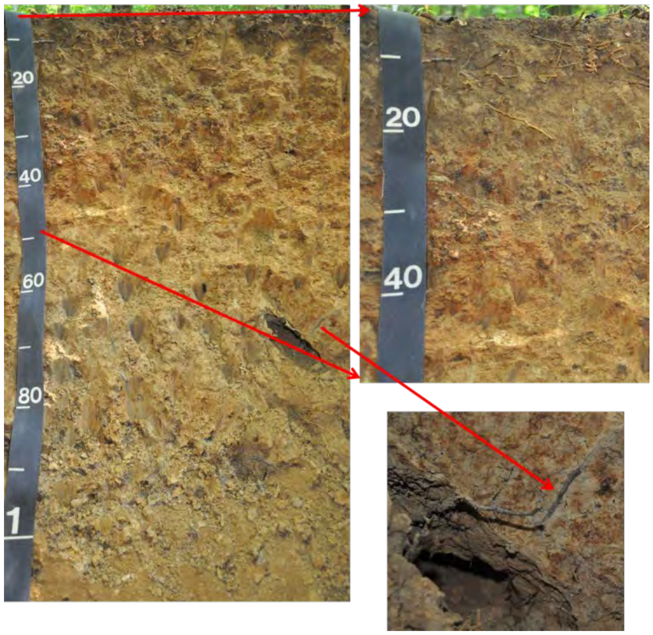

Figure 7: T1L5 – Chewacla Profile.

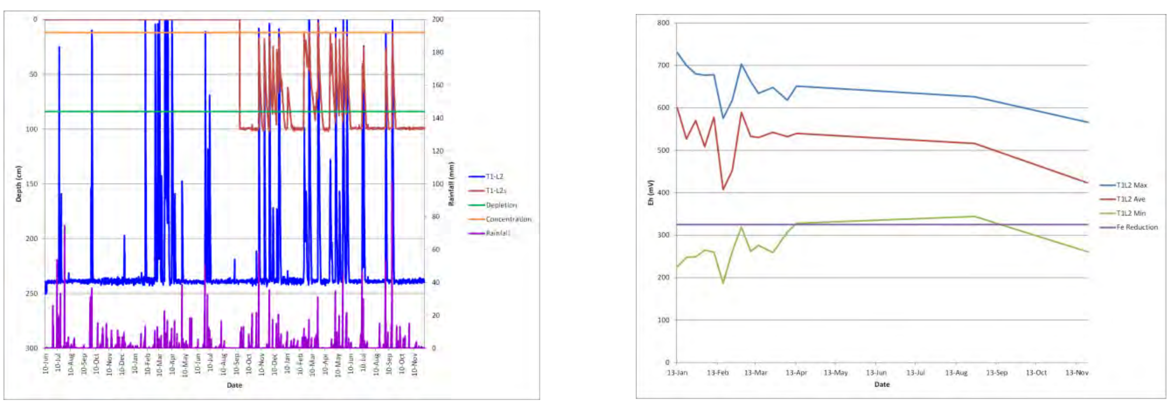

Figure 8: Hydrograph (left) and Eh (right) for Breeze Farm, Orange County location T1L2 (Georgeville) showing short duration spikes in the water table. The shallow hydrograph (red line) indicates that the spikes do correspond to saturation in the upper profile and that some episaturation does occur. The depletions are clearly related to the saturation observed in the shallow well but the entire horizon is not reduced. It is likely that reduction is localized.

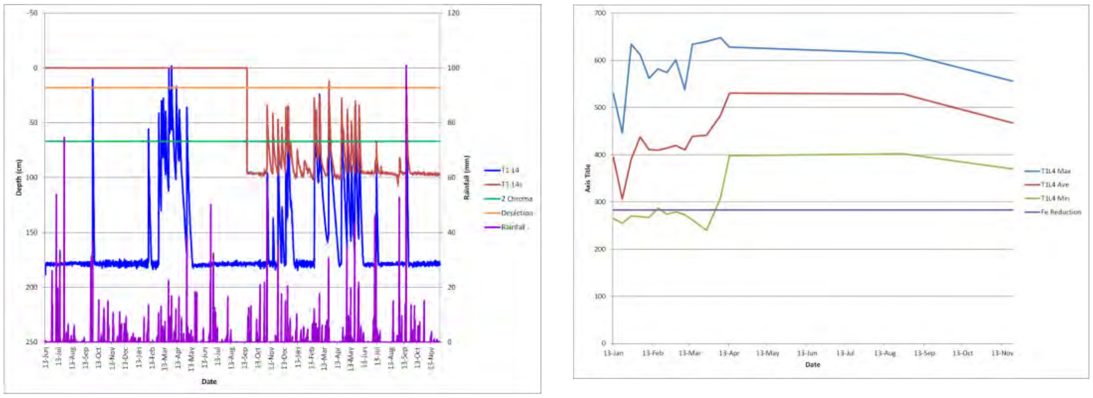

Figure 9: Hydrograph (left) and Eh (right) for Breeze Farm, Orange County location T1L4 (Lignum) showing more frequent and greater saturation than the middle Lignum location (T1L3 not shown). These hydrographs and morphology provide confirmation of the aquic conditions associated with this pedon. Despite the morphology the Eh does not confirm overall reducing conditions.

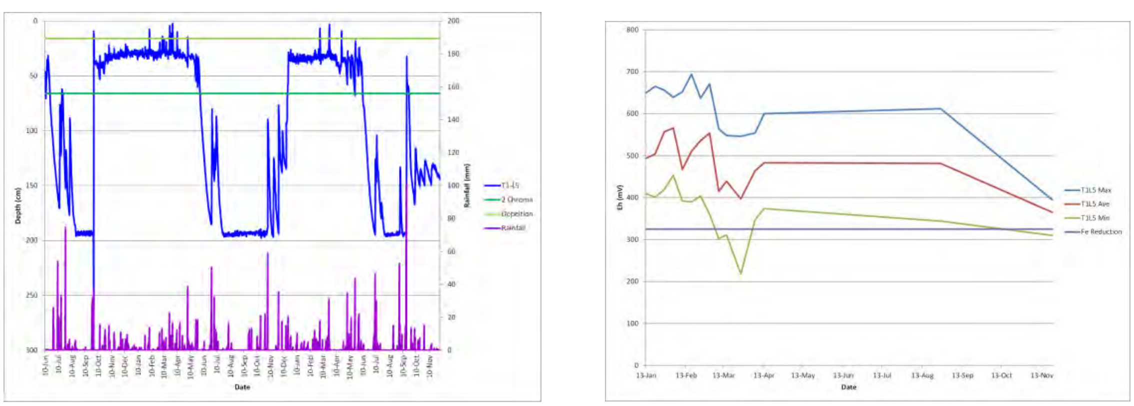

Figure 8: Hydrograph (left) and Eh (right) for Breeze Farm, Orange County location T1L5 (Chewacla) showing the depth to saturation during the winter to early spring at approximately 32cm. This is just below the depth of saturation commonly associated with hydric soils (30 cm). The high chroma depletions (light green line) appear to occur at depths that are rare saturated. As with the other pedons reduction was not observed in association with the observed depletions indicating this profile is not hydric.

Table 1: Summary of soil morphology at Breeze Farm Transects 1

Horizon	Depth	Tex	Matrix	Depletions	Concentrations	Structure	Cons.	Other
T1L1 – Georgeville S2012NC135001								
A	0-9	sicl	5YR 5/6			3mgr	fr,ss,sp	
Bt1	9-43	c	2.5YR 4/8			3cpr/2mabk	fi,ms,vp	
Bt2	43-68	sic	2.5YR 4/6		5YR 5/8	2cabk	fi,ss,mp	
BC	68-176	sicl	2.5YR 5/8		2.5YR 4/4 10YR 6/8	1vcabk	fr,ss,sp	
C	176-200	sil	5YR 6/8		10YR 6/8 2.5YR 4/6	m-rcf	fr,ns,sp	
T1L2 – Georgeville S2012NC135002								
A	0-12	sicl	7.5YR 4/6			2f-mgr	fr,ss,mp	
Bt1	12-46	sic	5YR 4/6		7.5YR 6/6	1cpr/3m-csbk	fi,ms,vp	
Bt2	46-84	sic	5YR 4/6		7.5YR 6/6	2msbk&abk	fi,ms,vp	
Bt3	84-173	sic	2-5YR 4/6	10YR 7/2	2.5YR 5/8 10YR 5/8	1mabk	fi,ms,vp	Depletions in channels
BC	173-200	sicl	2/5YR 4/6			1csbk	fr,ss,sp	
T1L3 – Lignum S2012NC135003								
A	0-10	sil	10YR 4/4			3mgr	vfr,ss,sp	
Bt1	10-38	sic	5YR 5/6		2.5YR 5/6 10YR 6/6	2m-cabk	fr,ms,mp	
Bt2	38-60	sic	7.5YR 5/6	10YR 6/1	2.5YR 5/6 10YR 6/6	2cabk	fr,ms,mp	Depletions in channels
Bt3	60-135	sic	7.5YR 5/6		10YR 6/6 2.5YR 5/6	2cabk	fr,ms,mp	Depletions in channels
BCt	135-200	sic	7.5YR 5/6	10YR 7/1	10YR 6/6 2.5YR 5/6	1cabk	fr,ss,sp	Depletions in channels
T1L4 – Lignum S2012NC135004								
A	0-6	sil	10YR 4/6			3cgr	vfr,ss,sp	
BA	6-18	c	5YR 4/4			2m-csbk	vfr,ss,sp	
Bt1	18-67	c	2.5YR 4/8		2.5YR 5/8 10YR 6/6	2cabk	fi,ss, vp	
Bt2	67-87	sic	2.5YR 5/6	10YR 7/2	10YR 6/6	2cabk	fr,ss, vp	Depletions in channels
Bt3	87-170	sic	7.5YR 5/6	10YR 7/2	2.5YR 5/6	2cabk	fr,ss, vp	Depletions in channels
BCt	170-185	sic	7.5YR 5/6	10YR 7/2	2.5YR 6/6	2cabk	fr,ss, vp	Depletions in channels
C	185-200	sil	10YR 6/6	10YR 7/2	7.5YR 4/6	m-rcf	fr,ss, sp	Depletions in channels
T1L5 – Chewacla S2012NC135005								
A	0-16	cl	10YR 5/4			2mgr	fr,ss, sp	
Bw	16-39	l	2.5Y 6/6	7.5YR 4/4		2msbk	fr,ss, np	
Ab	39-66	cl	10YR 4/6	2.5YR 6/4		1csbk	fi,ss, sp	
Bwb	66-148	c	2.5Y 6/8	2.5Y 7/1		2cabk	fi,ms,mp	
Bgb	148-175	c	2.5Y 7/1		10YR 6/6	2cabk		Fe-Mn concretions
Cg	175-200	scl	2.5Y 7/1		10YR 6/6	m		Fe-Mn concretions

Table 2: Saturated hydraulic conductivity measurements both *in situ* and laboratory.

Location	Depth							
	30cm	45cm	75cm	70-75cm (falling head)	105cm	45cm	75cm	105cm
	In situ (cm/hr)					Laboratory (cm/hr)		
T1L1		0.038	0.051		0.115	0.075	0.195	0.25
T1L2		0.084	0.03		0.018	0.303	0.192	0.17
T1L3		0.055	0.02	0.026	0.007	0.163	0.227	0.442
T1L4		0.122	0.023	0.035	0.018	0.236	0.385	0.312
T1L5		0.023	0.028	0.333		1.094	0.309	
T2L1			0.135					
T2L2			0.075					
T2L3			0.164					
T2L4			1.022					
T2L5	1.446							

Infiltrative Surface Clogging that Develops during Soil Treatment of Wastewater as Affected by the Interaction of Cations with Organic Matter

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ABSTRACT

Wastewater effluent infiltration into soil typically leads to genesis of a biozone at the infiltrative surface that results from biofilm formation, filtration of suspended solids, and accumulation of organic matter and microbial byproducts at the infiltrative surface and within the soil matrix. A set of experiments described in this paper enabled observations concerning the interactions of wastewater cations with the complex organic material that accumulates at the infiltrative surface. In the research, domestic septic tank effluent (STE) was applied to sand columns and *in situ* test cells in sandy loam soil. STE was applied for a period of about one year before chelating agents were introduced. As expected, STE application caused a dramatic loss in soil infiltrability (>90 to 99%). After this loss occurred, chelating agents (EDTA, EGTA, or citrate) were applied and observations were made concerning recovery of infiltrability and changes in soil pore water quality. In sand columns, application of chelating agents increased infiltration rates by an average factor of 12.2 for approximately three weeks, concurrent with increases in Al^{3+}, Ca^{2+}, Fe^{2+}, and Mg^{2+} in the pore water exiting the columns. In the test cells, citrate application increased infiltration rates by a factor of 2.7, and released Al^{+3} and Fe^{+2} cations into the soil pore water. EDTA application decreased infiltration rates by a factor of 0.94 within two weeks and mobilized Fe^{2+}. The results suggest that the bridging of organic molecules by polyvalent cations at and near the infiltrative surface can occur and might be a mechanism that contributes to biozone genesis and changes in infiltrability resulting from effluent infiltration during soil treatment of wastewater.

INTRODUCTION

Onsite and decentralized wastewater systems are widely used in the United States and abroad as a necessary and appropriate alternative to centralized treatment facilities. The most common system utilizes a septic tank followed by a soil treatment unit where septic tank effluent (STE) is infiltrated below ground surface into a native soil profile for tertiary treatment and natural disinfection before recharging local groundwater (Siegrist, 2008). STE infiltration into soil typically leads to genesis of a biozone at the infiltrative surface that results from biofilm formation, filtration of suspended solids, and accumulation of organic matter and microbial byproducts at the infiltrative surface and within the soil matrix (Siegrist et al., 2001; Lowe and Siegrist, 2008; McKinley and Siegrist, 2011). A wastewater-induced biozone can be beneficial by providing a more biogeochemically active zone and helping to maintain an unsaturated flow regime in the soil profile. However, if soil clogging becomes too intensive it can reduce the infiltrability below the operational STE loading rate and cause hydraulic malfunctions.

Previous research has suggested two inter-related mechanisms responsible for wastewater-induced soil clogging (e.g., Siegrist, 1987; Siegrist et al., 2001; McKinley and Siegrist, 2011). The first mechanism involves the accumulation of suspended solids at the infiltrative surface by filtration while the second includes enhanced microbial growth and the resultant accumulation of microbial byproducts. As nearly two thirds of the filtered solids can be organic (Jawson, 1976) their deposition can stimulate microbial growth at the infiltrative surface. In addition, the dissolved organic carbon and nutrients in the STE can stimulate further growth and the accumulation of microbial byproducts at the infiltrative surface and within the soil pores. Previous research has identified these byproducts as polysaccharides and humic substances (HS)

(Siegrist et al., 1991; McKinley and Siegrist, 2010). This paper explores a third inter-related mechanism potentially involved in wastewater induced soil clogging: the interaction of polyvalent cations in STE with the negatively charged functional groups on polysaccharides and HS present at the infiltrative surface and within the soil matrix.

Cations are known to interact with polysaccharides and HS, comprised of humic acid (HA), fulvic acid (FA), and humin. Polysaccharides and HS contain functional groups, of which the most reactive are the carboxylic (-COOH) and phenolic (-OH) groups (Schnitzer and Khan, 1978). The negative charges of these functional groups are variable and pH dependent, and can attract cations from solution that can adsorb to the organic molecule through electrostatic interactions (Tan, 2003). If the sorbing cation is polyvalent (Al^{3+}, Ca^{2+}, Fe^{2+}, or Mg^{2+}), the organo-cation complex has a partial positive charge, which may attract other negatively charged molecules (inorganic or organic), creating a bridge between the two negatively charged substances. In soil environments cation bridging between organic and inorganic molecules is responsible for the formation of soil aggregates, which are important to soil stability (Chaney and Swift, 1984). Aggregation of soil improves its ability to transmit water and oxygen, which is beneficial for soil and plant health (Hamblin and Davies, 1977). Complex organic materials with modified benzene rings, such as HA, can be strongly bridged with mineral particles when polyvalent cations are present, resulting in more stable aggregates with better hydraulic properties (Gu and Doner, 1993).

The electrostatic interactions responsible for soil aggregation can also result in biofilm clogging when two negatively charged organic molecules are bridged by a multivalent cation (Donlan, 2002). Researchers have observed that in membranes from centralized water treatment facilities, when multivalent cations are present in the water, a stronger, more cohesive, and less permeable biofilm is formed (Lee and Elimelech, 2006). Although any polyvalent cation can serve as a bridge, Ca^{2+} was most commonly correlated with reductions in biofilm permeability and membrane fouling. Further research focused on appropriate technologies for removal of bridging cations and biofilms from fouled membranes (Ang et al., 2006). This research revealed that the removal of bridging cations from a biofilm through the application of a chelating agent resulted in a less dense, more permeable, and more soluble biofilm structure, thus decreasing membrane fouling.

In the research described above, the bridging cations were removed with the aid of chelating agents, such as ethylenediamine tetraacetic acid (EDTA), ethylene glycol-bis(2-aminoethyl ether)-N,N,N',N'-tetraacetic acid (EGTA), and citrate. These same chelating agents have also been used to mobilize heavy metals in soils (Jackson and Larkins, 1976; Heil et al., 1996; Lesage et al., 2005). EDTA is more recalcitrant than EGTA or citrate, while EGTA demonstrates a higher affinity for Ca^{2+} than other cations, and citrate is a naturally occurring biodegradable chelating agent. Although considerable research has been conducted using chelating agents to remove biofilms involved in clogging of membranes used in centralized water treatment facilities, these agents have not been previously applied to soil infiltrative surfaces with wastewater-induced clogging and reduced infiltrability.

MATERIALS AND METHODS

Experimental Approach. The research was conducted at the Mines Park Test Site located on the Colorado School of Mines (CSM) campus in Golden, Colorado. At the Test Site, wastewater from an 8-unit multi-family apartment building is collected and processed in two 5,678-liter

(1,500-gallon) septic tanks in series prior to use in field and laboratory research (e.g., Tackett et al., 2004; Lowe and Siegrist, 2008). Pertinent to the research described in this paper, STE was applied to sand columns established in a field laboratory and *in situ* test cells installed in Ascalon sandy loam soil.

The sand columns were established at the Test Site and previously used to examine infiltrability loss as affected by effluent quality, loading rate, and infiltrative surface architecture (Walsh, 2006). Clear acrylic columns (16-cm diam.) were packed with a 55-cm layer of medium to coarse sand (d_{10} = 0.21 mm, d_{60} = 0.63 mm, TOC = 0.0 wt. %, pH = 8.92) yielding a saturated hydraulic conductivity after packing that averaged 4,484 cm/d (coefficient of variation = 10.8%). STE was applied to the columns at a rate of 20 cm/d, and infiltration rates (IRs) were determined daily. After one year of STE loading, the IRs of the columns had decreased by >99% due to soil clogging at the infiltrative surface. Four of the clogged sand columns were used in the research described in this paper.

The test cells were constructed at the Test Site to simulate a typical trench design used for subsurface infiltration in an onsite wastewater treatment system. A set of test cells had been installed and used in prior research to examine infiltrability loss as affected by hydraulic loading rate and infiltrative surface architecture (Tackett, 2004; Lowe and Siegrist, 2008). Each test cell was constructed approximately 76 cm below ground surface in Ascalon sandy loam soil (TOC = 0.5-1.0 wt. %, pH = 7.3), with an average horizontal infiltrative surface area of 5,385 cm^2 (Lowe and Siegrist, 2008). Prior to STE loading, the average clean water IR measured under a constant head of 2.5 cm was 41.8 cm/d (std. dev. = 20.8 cm/d). STE was applied to the test cells at 4 cm/d for 11 months prior to this study, resulting in reductions in hydraulic capacity and ponding of STE at the infiltrative surface. In the research described in this paper, IRs were calculated weekly by measuring the rate of decrease in STE ponding from 5.1 to 2.5 cm above the infiltrative surface through observation ports. The average IR was determined by linear regression of the decrease in ponding over time (R^2 >95%). During the 11 months of STE loading, the IRs of the test cells decreased by >90% due to soil clogging at the infiltrative surface. Seven of these clogged test cells were used in the research described in this paper.

Chelating agents including sodium EDTA, sodium EGTA, and potassium citrate were applied to the sand columns and test cells to observe the hydraulic response and to determine the extent of polyvalent cation bridging as a possible mechanism for wastewater-induced clogging in porous media. Chelating agent solution concentrations were based on previous research that used chelating agents to remove biofilms from membranes used in water treatment facilities or to mobilize heavy metals in soils. A 50 mM sodium EDTA concentration was adapted from Heil et al. (1996), a 20 mM sodium EGTA concentration was based on Jackson and Larkins (1976), and a 100 mM potassium citrate concentration was based on Lesage et al. (2005). To account for the difference in number of complexation sites (citrate forms three bonds with cations; EDTA and EGTA form six), the citrate solution was more concentrated than the EDTA and EGTA solutions. A summary of the application sequence can be found in Table 1 while further details may be found elsewhere (McKinley, 2008). Grab samples of STE were collected and analyzed biweekly during experimental use for physical, chemical, and biological parameters. The characteristics of the STE were within normal ranges for residential applications

<u>Chelating Agent Application and Response Monitoring.</u> Prior to chelating agent application in the sand columns, the infiltrative surfaces had been continuously ponded with STE for 12 mon. Immediately prior to chelating agent application, the columns were taken offline and the

ponded effluent was pumped down to within 1 cm of the infiltrative surface. Chelating agent solutions were applied the following day (corresponding to experimental day 8) when no ponded STE was observed above the infiltrative surface. A volume of EDTA, EGTA, or citrate solution (3.8 L) was applied to each of three columns, and 3.8 L of deionized water (DI) was applied to a fourth column as a control. After the entire volume of chelating agent solution had infiltrated and exited as percolate (after 1-2 days), the columns were put back online and loaded with STE at 20 cm/d. Seventy-five days after the first application, chelating agents and DI water were again applied to observe cumulative effects of chelating agent applications and whether more cations could be removed. Citrate was applied to the same column; EDTA was applied to the original EGTA column, and vice versa, to determine if cations other than Ca^{2+} were responsible for organic material bridging and clogging. IRs were measured daily during a 7-day pre-application period to determine baseline values, and then measured post-application for the duration of the 108-day experiment. Immediately after each chelating agent application, percolate was collected from the columns until the approximate volume of chelating agent applied was recovered. The percolate was analyzed for cations, including Al^{3+}, Ca^{2+}, Fe^{2+}, and Mg^{2+}, using an inductively coupled plasma procedure with a Perkin Elmer Optima 3000 ICP atomic emission spectroscope (McKinley, 2008).

EDTA and citrate were applied to three clogged soil test cells each; in the same concentrations used in the sand columns and DI water was applied to one test cell. EGTA was not used in the test cell work based on observations made during the sand column study and the high cost of the chemical and a perceived limited practicality for field use. Prior to application, the pH of the chelating agent solutions was adjusted to pH 7 by adding small amounts (several ml) of concentrated HCL or NaOH. This pH adjustment was done so that the microorganisms that contribute to treatment in sandy loam soil would not be harmed by excessively high or low pH. Before application of the chelating agents, STE loading to test cells was suspended to allow the ponded effluent to infiltrate into the soil (<1 day), after which time 30.3 L (approximately 5 cm depth) of chelating agent solution was applied with a peristaltic pump from a height of 2.5 cm above the infiltrative surface. Once the chelating agents were completely infiltrated and no ponding remained (1 day to 3 weeks), the test cells were returned to operation at the original STE loading rate of 4 cm/d. IRs were measured weekly for three weeks prior to application of chelating agents to determine baseline values, and weekly for five weeks post-application.

During initial installation of the test cell network, a subset of the test cells was outfitted with stainless steel microporous suction lysimeters for soil pore water sampling (Model SW-074, pore size = 0.2 μm, Soil Measurement Systems of Tucson, Arizona). One of the test cells that received the EDTA application and one that received the citrate application contained lysimeters. Soil pore water samples were collected from the lysimeters at 60-cm depth below the infiltrative surface one week prior to chelating agent application, continuously after application for 10 days in five 2-day sampling events, and again approximately six weeks after application. Soil pore water sampling followed the methodology used previously at the test site. To ensure that these sampling events captured the pulse of the chelating agent as it percolated through the soil profile, the timing of the sampling events was adjusted to reflect the vadose zone transport rates determined with bromide tracer tests conducted during the installation of the test cells (Tackett, 2004; Lowe and Siegrist, 2008). Samples were analyzed for cations, including Al^{3+}, Ca^{2+}, Fe^{2+}, and Mg^{2+}, following the same procedure used for the sand column percolate; dissolved organic carbon (DOC) (Sievers 5310C analyzer); and ultraviolet absorption (Beckman Coulter DU 800 spectrophotometer).

RESULTS AND DISCUSSION

IR Response and Percolate Water Quality. The pre-application baseline column IRs ranged from 1.5 to 3.4 cm/d. The first application of chelating agents on day 8 increased column IRs by an average factor of 12.2 (EDTA = 23.7; EGTA = 5.6, citrate = 7.4), while the application of DI water increased the IR by a factor of 3.7 (Fig. 1). The increases in IR lasted for approximately three weeks, at which time the IRs in all treated columns declined to baseline values. At day 75, chelating agents were again applied to the columns. The second application of chelating agents increased the column IRs to values slightly below those achieved during the first application, and the increase had a shorter duration than that achieved by the first application. The only exception was column 2, which received EGTA for the first application and EDTA for the second. In that column, the second increase in IR was higher than the first increase. Based on percolate water quality analyses (McKinley 2008), all three chelating agents were successful in mobilizing high concentrations of cations. Citrate generally mobilized higher concentrations of cations, followed by EDTA and EGTA. For the second application of chelating agents, citrate demonstrated less mobilization of Al^{3+} and Fe^{2+}, although Ca^{2+} and Mg^{2+} were comparable to the first application. When EDTA was applied to the column previously infiltrated with EGTA, high concentrations of Al^{3+} and Fe^{2+} were mobilized, corresponding to the largest increase in column IR observed during the second application.

In the soil test cells, citrate application increased the IRs by an average factor of 2.7 (1.8-3.6) (Fig. 2). After STE application was restarted, the IRs decreased to or below pre-application rates within two to five weeks. All EDTA applications decreased IRs by an average factor of 0.94 (0.89 to 0.97) within two weeks. The application of DI water increased the IR by a factor of 0.24, which subsequently resumed to pre-application values within three weeks. Al^{3+}, Ca^{2+}, Fe^{2+}, and Mg^{2+} were present in lysimeter samples collected from the test cell receiving citrate, although Ca^{2+} and Mg^{2+} were present at concentrations near background levels (McKinley, 2008). Based on pore water sampling, Al^{3+} and Fe^{2+} were present at concentrations significantly higher than background values (near zero) and remained high for the duration of the 10-day period immediately after chelating application. When the lysimeter was sampled six weeks after chelating application, high concentrations of Al^{3+} and Fe^{2+} were no longer present. The EDTA application did not mobilize Al^{3+}, Ca^{2+}, or Mg^{2+} at concentrations higher than baseline values. This result differs from the mobilization of Al^{3+} observed during the column work. However, EDTA was effective in mobilizing Fe^{2+} at higher concentrations (200% higher) and for a longer duration (12 weeks) than the citrate application.

Analyses of carbon concentrations in the lysimeter samples were completed to provide insight into potential reasons for the immediate hydraulic failure of the test cells after EDTA application. Specific ultraviolet absorption (SUVA) values were calculated by dividing the DOC of the sample by its ultraviolet absorbance at 254 nm. Typically, more complex and recalcitrant organic materials, such as HA, contain modified benzene rings that exhibit high SUVA values (4.8 to 7.4 m-L/mL). Less complex and recalcitrant organics, such as FA and polysaccharides, contain more chain structures and fewer rings and exhibit lower SUVA values (2.9 to 4.3 m-L/mL) (Musikavong et al., 2005). Measurements in lysimeter samples revealed that EDTA application resulted in substantially higher SUVA values (6 m-L/mL) compared to citrate application (0.5 to 1.56 m-L/mL). EDTA application thus appeared to be effective in removing HA from the sandy loam soil while the citrate application was not.

Interactions of Cations with Organic Matter. In the sand columns studied, after the initial increase in IRs following the first application of chelating agents, IRs decreased back to baseline values within three weeks after STE application resumed. This IR decrease may have been caused by the complexation of STE cations with the newly vacant functional groups on the organic material at the infiltrative surface, thereby decreasing the infiltrability. The decrease may also indicate that other clogging mechanisms, such as the filtration of wastewater solids and the accumulation of microbial byproducts, were dominant at and immediately below the infiltrative surface of the sand. After the second application of chelating agents to the sand columns on day 75, the IR increase lasted for a significantly shorter time than after the first application across all columns. In column 3, citrate mobilized less Al^{3+} and Fe^{2+} compared to the first application. The decreased mobilization may help explain why the increase in IR was less pronounced after the second application; i.e., there were fewer bridging cations present because they had been removed during the first application. In column 2, the EDTA application on day 75, which followed the prior EGTA application on day 8, mobilized high concentrations of Al^{3+} and Fe^{2+}, and resulted in the highest increase in IR observed during either application of chelating agents. Although cation-bridging research has focused on Ca^{2+} (e.g., Lee and Elimelech, 2006), this result demonstrates that other polyvalent cations (Al^{3+} and Fe^{2+}) may have equal importance in bridging in wastewater-affected environments. The mobilization of Al^{3+} and Fe^{2+} was unexpectedly high, considering the relatively low concentrations of these metals in STE and in the baseline column percolate (<1 mg/L for both cations). These results demonstrate that substantial concentrations of polyvalent cations accumulate in the infiltrative surface zone and within the porous media matrix, even when these cations are present in STE at low concentrations.

In general, citrate mobilized higher concentrations of cations from the columns than EDTA or EGTA. This result was unexpected because of the lower intrinsic stability constants of citrate complexes (Table 2). However, intrinsic stability constants (K_i) do not account for the pH of the environment. Conditional stability constants (K_c) (Table 2) were calculated for citrate, EDTA, and EGTA complexes at the percolate pH values (3.8, 3.89, and 3.5, respectively) using equations provided in Harris (2006). Although the K_c value for a citrate complex with Al^{3+} is higher than the stability constants for EDTA or EGTA complexes, the constants do not explain why citrate complexed higher concentrations of Ca^{2+}, Fe^{2+}, and Mg^{2+}. A possible explanation for the relative efficiency of citrate to complex cations is that the citrate solution was more concentrated (100 mM) than the EDTA (50 mM) or EGTA (20 mM) solutions. EGTA generally exhibited the lowest concentrations of complexed cations.

In the soil test cells, after the initial increase in IR values following citrate application, the IRs decreased to pre-application values within two to five weeks of STE application. This decrease may have been caused by the complexation of STE cations with accumulated organic material, as noted above. The decrease may also have been caused by the citrate itself, which contains a high concentration of biodegradable carbon and may have stimulated microbial clogging after the initial IR increases. Conversely, after EDTA application, IRs decreased an average of 94%, which was markedly different from the IR increase observed in the sand columns after EDTA application. EDTA biodegrades slowly relative to citrate, so it is unlikely that the EDTA application stimulated microbial clogging.

There were two major differences between the soil test cell and sand column experiments, which may explain the contrasting EDTA results. The first difference is that the chelating agents were adjusted to pH 7 before application to the soil test cells, but not before application to the sand columns. Different pH values result in different K_c values, and were calculated for citrate- and EDTA-Al^{3+} complexes in the column percolates and lysimeter samples to determine why citrate was effective at chelating Al^{3+} in sand columns and test cells, but EDTA was only effective in the sand columns **(**Table 2**)**. In the EDTA and citrate sand column percolates and citrate lysimeter samples, EDTA- and citrate-Al^{3+} complexes had similar K_c values. By comparison, the K_c for EDTA-Al^{3+} in the lysimeter samples was over two orders of magnitude weaker. This difference may explain why Al^{3+} was not observed in the samples from the test cell infiltrated with EDTA.

The second major difference between the experiments is that the columns contained homogenous, clean sand, while the test cells were installed *in situ* in a sandy loam soil. Sand does not require organic material to maintain its pore structure and permeability. However, in soil, the formation of aggregates can be vital in maintaining soil structure and hydraulic properties. Although EDTA was not effective in mobilizing Al^{3+} in the test cells, as explained by the low K_c values, EDTA was effective in mobilizing high concentrations of Fe^{2+} for a much longer time period compared to citrate. This result can be explained by the difference in K_c values for Fe^{2+} complexes with citrate (14.21) and EDTA (18.49) at the pH values observed in the lysimeter samples. EDTA-Fe^{2+} complexes are more than four orders of magnitude stronger than citrate-Fe^{2+} complexes, and more than 14 orders of magnitude stronger than the K_i between natural organic matter and Fe^{2+} (4.3). Citrate may have complexed Fe^{2+} that was weakly bound in bridged organic complexes, but EDTA may have removed both weakly bound Fe^{2+} responsible for organic bridging and strongly bound Fe^{2+} that bridged organic matter and inorganic material in soil aggregates. The complexation of strongly bound soil structural cations by EDTA may explain the higher Fe^{2+} concentrations and SUVA values observed in the EDTA lysimeter samples, and the immediate 94% decrease in IRs.

CONCLUSIONS

The application of chelating agents to sand media and sandy loam soil that had experienced wastewater-induced clogging due to STE application, resulted in mobilization of polyvalent cations and a concurrent increase in infiltrability. When different chelating agents were applied to test cells installed in sandy loam soil, the results varied from those observed during application to sand columns. This is thought to be due to differences in the chelating agents themselves and how they can stimulate microbial activity or disrupt native soil structure, both of which can affect infiltrability. The results of this research suggest that, as a result of wastewater effluent infiltration into soil, bridging of organic material with polyvalent cations can occur at the infiltrative surface and immediately below it. However, the degree to which cation bridging of organic molecules contributes to biozone genesis and wastewater-induced soil clogging is unclear. Further, this potential mechanism does not act independently of the other mechanisms involving solids filtration and microbial byproduct formation. The significance of cation bridging and its effects on long-term soil treatment of wastewater under different design and environmental conditions is not yet fully understood and remains a topic requiring further research.

ACKNOWLEDGEMENTS

Funding for this research was provided by the CSM Small Flows Program. Assistance with the experimental work was provided by CSM Professor, Dr. Patrick MacCarthy, and CSM researchers Kathy Conn, Jill Tomaras, Dean Heil, and Christiane Hoppe. Assistance with compiling and organizing the data presented in this paper was provided by Rebecca Parzen.

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Table 1. Summary of the experimental methods.

Sand column (SC) or test cell (TC) no.	IR_B[2] (cm/d)	Chelating agent application[1]				IR measurements[2]	Percolate sampling[3]
		1st	2nd	Vol. and depth	Time to infiltrate (d)		
SC1	0.7	EDTA	EGTA	3.8 L 20 cm	2	Daily during pre- and post-application	Pre-application and 1, 2, or 5 days post-application
SC2	3.4	EGTA	EDTA		1		
SC3	2.4	Citrate	Citrate		1		
SC4	1.5	DI	DI		5		
TC1	6.7	EDTA	-	30.3 L 5 cm	7	Weeky during pre- and post-application	Pre-application, continuously for 10 days post-application and again after 6 weeks
TC2	3.1	EDTA	-		21		
TC3	10.0	EDTA	-		7		
TC4	3.8	Citrate	-		2		
TC5	3.4	Citrate	-		2		
TC6	6.7	Citrate	-		1		
TC7	4.5	DI	-		1		

[1] EDTA = ethylenediamine tetraacetic acid, EGTA = ethylene glycol-bis(2-aminoethyl ether)-N,N,N',N'-tetraacetic acid. Chelating solution concentrations: EDTA = 50mM, EGTA = 20mM, Citrate = 100mM. [2] Baseline IRs (IR_B) were measured daily for 7 days prior to chelating agent application to the SC and weekly for 3 weeks prior to application to the TC. [3] Soil pore water samples were collected from lysimeters at 60 cm depth in TC1 and TC4.

Table 2. Stability constants for complexes between chelating agents and cations.[1]

Agent	Agent formula and MW	Cation	Intrinsic stability constant (Ki)	Log conditional stability constant vs. pH (Kc)		Log conditional stability constants at pHs observed in pore water (Kc)	
						Sand columns	Test cells
EDTA	$C_{10}H_{16}N_2O_8$ 372g/mol	Al^{+3}	16.1	pH 3.9	4.27	4.27 @pH 3.89	1.18 @pH 6.50
		Ca^{+2}	10.7		27.06		
		Fe^{+2}	14.3		22.37		
		Mg^{+2}	8.7		19.10		
EGTA	$C_{14}H_{24}N_2O_{10}$ 380g/mol	Al^{+3}	14.3	pH 3.5	0.86		
		Ca^{+2}	11.0		25.83		
		Fe^{+2}	11.8		17.55		
		Mg^{+2}	5.2		5.09		
Citrate	$C_6H_5K_3O_7$ 324g/mol	Al^{+3}	11.7	pH 3.8	5.10	5.1 @pH 3.80	3.57 @pH 5.00
		Ca^{+2}	3.5		25.12		
		Fe^{+2}	3.2		15.73		
		Mg^{+2}	2.8		18.47		
Natural organic matter	-	Al^{+3}		-	3.75		
		Ca^{+2}	-		3.27		
		Fe^{+2}	-		4.30		
		Mg^{+2}	-		2.36		

[1] For details on calculations of the constants shown refer to McKinley (2008). EDTA = ethylenediamine tetraacetic acid, EGTA = ethylene glycol-bis(2-aminoethyl ether)-N,N,N',N'-tetraacetic acid.

Figure 1. Infiltration rate responses to two applications of chelating agents in sand columns. (Note: The first round of applications began on day 8 and the second round began on day 75. Average infiltration rates and standard deviations were calculated prior to the first application (baseline) and post-application (days 8-33) for each treatment: DI baseline = 1.5 ± 0.2; DI post-application = 5.5 ± 1.9; EDTA baseline 0.7 ± 0.3; EDTA post-application 16.6 ± 5.7; EGTA baseline 3.4 ± 1.4; EGTA post-application 18.9 ± 6.5; Citrate baseline 2.4 ± 0.8; Citrate post-application 17.8 ± 1.8.)

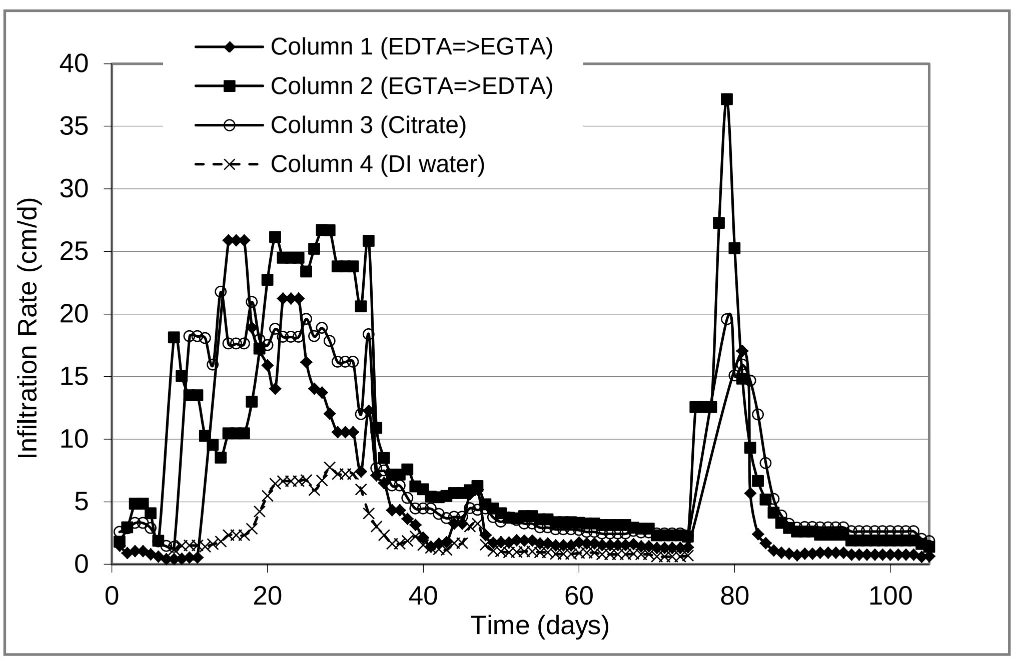
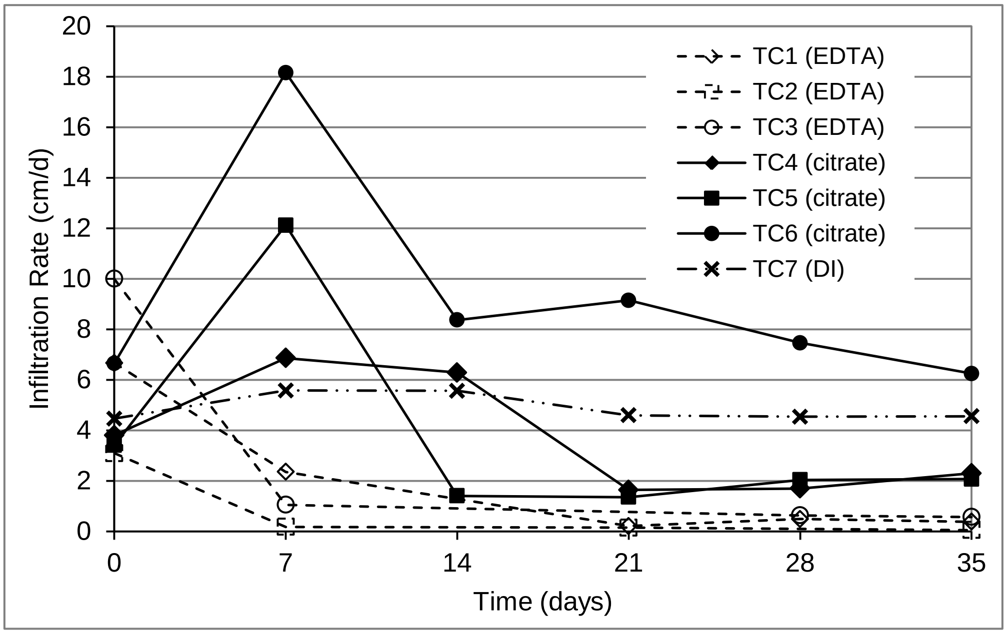


Figure 2. Infiltration rate responses to application of chelating agents in test cells in sandy loam soil. (Note: The baseline values shown at day 0 are an average of three IRs measured weekly prior to chelating agent application (average std. dev. = 0.7). The five subsequent IR measurements for Test Cells 1, 3, 4, 5, 6, and 7 were taken weekly after the infiltration of the chelating agent solution into the test cell had completed and STE loading had resumed. For Test Cell 2, the IR was measured on the infiltration of the chelating agent solution on day 7 because the solution had not yet infiltrated into the test cell; measurements on days 21 and 35 were taken after STE loading had been resumed.)



Oxygen Transfer and Clogging in Vertical Flow Sand Filters for On-Site Wastewater Treatment

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ABSTRACT

13 million people (about 20% of the population) use on-site wastewater treatment in France. Buried vertical sand filters are often built, especially when the soil permeability is not sufficient for septic tank effluent infiltration in undisturbed soil. Clogging is one of the main problems deteriorating the operation of vertical flow filters for wastewater treatment. The extent of clogging is not easily assessed, especially in buried vertical flow sand filters. We suggest examining three possible ways of detecting early clogging: (1) NH_4-N/NO_3-N outlet concentration ratio; (2) direct measurement of oxygen content within the porous media. This information can be obtained by gas analysis of the filter's air phase; and (3) outflow volume measurements. Two pilot-scale filters were equipped with probes for oxygen concentration measurements and samples were taken at different depths for pollutant characterization. Influent and effluent grab-samples were taken three times a week and analyzed using standard methods. The systems were operated using batch-feeding of septic tank effluent. After a starting period of 6 weeks (average load: 30 g_{COD}/m²/d and 8 g_{SS}/m²/d), clogging was induced using an average daily pollutant load of 55 g_{COD}/m²/d and 12 g_{SS}/m²/d, resulting in permanent ponding between two successive batches. Qualitative description of oxygen transfer processes under unclogged and clogged conditions is presented. Nitrate outlet concentration and oxygen concentration inside the filter appear to be better parameters for diagnosing clogging in vertical flow filters than ammonium outlet concentration and outflow values.

Vertical unsaturated subsurface flow filters are widely used for the treatment of wastewater, whether from individual houses or small communities. The main cause of malfunction of these systems is clogging, which is the temporary or definitive reduction of their hydraulic conductivity. Severe cases reduce the effectiveness of systems to the extent that treatment is no longer feasible. Knowles et al. (2011) listed the main causes leading to clogging: (1) solid entrapment, linked to SS retention; (2) biofilm growth, due to biological activity and hence to biodegradation; (3) vegetation growth, the effects of which have not yet been clearly defined; (4) chemical effects, consisting mainly of precipitation and adsorption. It is important to understand that the processes involved in the development of clogging are precisely those required for wastewater treatment. Therefore, it is not advisable to eliminate the causes of clogging, but rather to distinguish between normal operation and pathological clogging. Design and operational characteristics that influence the occurrence of clogging include: (1) wastewater characteristics (Winter and Goetz, 2003); (2) loading rate (Langergraber et al., 2003) and frequency; (3) media characteristics (Lowe et al., 2008); and (4) the type of inlet distribution system (Pavelic et al., 2011). All these considerations also impact oxygen transfer in vertical flow filters, that is an observable parameter (Platzer and Mauch, 1997, Rolland et al., 2009) linked to the clogging phenomenon.

The problem of distinguishing between normal operation and pathological clogging is particularly acute for systems that do not offer access to the surface of the filter, as is the case in on-site wastewater treatment systems. Consequently, clogging issues are often discovered late, when the system is completely blocked, leaving no other option than its replacement. In this paper, we suggest examining three possible ways of detecting early clogging: (1) NH_4-N/NO_3-N outlet concentration ratio. This ratio depends on the nitrification process occurring in the filter. High values mean low nitrification and subsequent poor oxygenation of the filter that might be clogged; (2) direct measurement of oxygen content within the porous media. A low value means that the filter might be encountering re-oxygenation problems depending on the degree of filter clogging; and (3) outflow volume measurements. Clogging affects the overall hydraulic conductivity of the filter. Therefore, the outflow volume is modified when clogging occurs.

MATERIALS AND METHODS

The pilot-scale filter is a 1-m high column, with diameter of 0.4 m. This study is based on the assumption of one-dimensional flow in vertical filters: neither water nor solute transport in the horizontal plane is considered. The aim of the system is to describe processes occurring in a localized section of field-scale filters. The active layer (60 cm) is composed of 0 – 4 mm alluvial sand (d_{10} = 0.16 and CU = 0.3, fines < 3 %). A 10-cm layer of 10 – 20 mm gravel covered the sand layer in order to ensure equal distribution of the effluent. A 10-cm layer of the same gravel was put underneath the sand layer in order to drain the water (seepage face). Two of these pilot-scale filters were used to test the repeatability of the results.

The weights of the influent and effluent tanks, as well as the column masses, were monitored using scales (NOBEL, France). The uncertainty on weight was below 0.05% for their respective ranges (50 - 300 kg and 1 - 60 kg for the column and tank balances, respectively). The outflow value could be computed by deriving either the weight of the column or of the effluent tank. PT100 probes were used to measure the temperature in the core of the filter and in the influent tank. All data were recorded at 1-min intervals using a data logger (Gantner, Austria).

The filter was equipped with oxygen optical probes (PreSens, Germany), intended to measure the oxygen content in both the water and air phases. These probes are small, consume hardly any oxygen, have a response time of about 1-min in water, and have excellent long-term stability, providing a few months of continuous operation without needing re-calibration. They have been used successfully in sand filters by Turković and Fuchs (2010) and Wozniak et al. (2007). The oxygen concentrations are expressed as a percentage of the oxygen saturation, whether in the water or air phase. Oxygen content profiles are established from 10 measurements, using spatial linear interpolation.

Influent and effluent grab-samples were taken three times a week and analyzed using standard methods (APHA, 2005). The outlet tank was big enough to hold the daily outflow, and was emptied after the daily averaged samples had been taken. The parameters measured were COD, SS, total Kjeldahl nitrogen (TKN), NH_4-N, NO_2-N, and NO_3-N (concentrations). The value of uncertainty for the measurements is about 5% for COD, TKN, NH_4-N, NO_3-N, NO_2-N, and 10 % for SS.

Figure 1 gives pilots loading strategy. The filters were intermittently loaded (3 or 4 times a day) with septic tank effluent from a household of 5 persons (3-m^3 septic tank capacity). The effluent was applied to the filters via peristaltic pumps, and every batch led to ephemeral ponding, thus ensuring equal distribution on the filter's surface. The first part of the experiment concerned the inoculation of the filters. Pollutant load was similar the one used for vertical flow sand filters for on-site wastewater treatment in France (septic tank effluent). For the second part of the experiment the hydraulic and pollutant loads were increased in order to obtain the first clogging event. In this study, clogging was defined as constant ponding on the surface of the filters between 2 consecutive batches.

RESULTS AND DISCUSSION

Figure 2 shows the mass of ammonium, TKN, and nitrate at the inlet and outlet. NO_3-N concentrations in the influent were always under 0.45 mg N/l. The following sequence of events is suggested by the data: Phase 1 occurred at the very beginning of the experiment. The filter was clean and constantly loaded with doses of TKN and COD. Neither TKN nor COD outlet fluxes were observable. Phase 1 ended when nitrate started coming out of the column, indicating the beginning of the nitrification process. Phase 2 represents the normal operation of vertical filters, i.e. nitrification and organic carbon oxidation. During this phase, the outlet flux of NO_3-N indicated that about 70% of the TKN underwent nitrification. The outlet TKN flux was very low. The outlet COD concentration was below or close to the limit of quantification (30 mg/l). During Phase 2, a periodical, unsteady operation of the filters was attained. This means that organic and ammonium loads were treated or stocked in the filter within the batch, while conditions after the batch were good enough to allow the handling of the subsequent batch. After a 25-day period of normal operation of the filters, it was decided to increase the hydraulic and pollutant loads in order to clog the filter (see Fig. 1). Phase 3 started after this change. A lack of oxygen stopped the nitrification process (no more NO_3-N outlet flux). NO_2-N concentrations were tested periodically during the experiments and did not increase during this period (<0.02 mg N/l). A breakthrough of NH_4-N appeared only 9 days after nitrification had stopped. COD outlet concentrations (not shown) were not significantly altered: only a slight increase was observed, which remained close to the limit of quantification (mean value of 40 mg/l). The clogging event could not be foreseen by observing the ammonium outlet concentrations, which increased only 9 days after the beginning of the event. However, nitrate concentration decreased 1 day before the beginning of clogging, and could indicate the need for filter monitoring for this parameter.

Figure 3 displays both the mass of the filter and the oxygen concentrations versus depth and time for Phases 2 and 3. Oxygen concentration is expressed as a percentage of the oxygen saturation. The reference depth is chosen at the gravel-sand interface. At reference depth, oxygen saturation (100% $[O_2]_{sat}$) was always assumed, except when ponding was observed (then the chosen value is 0%, which is the oxygen concentration in the influent). Ephemeral ponding was observed after every batch. Constant ponding between two batches appeared when the filters were nearly clogged. Initially, an evolution of oxygen content due to convective and diffusive flux in the filter is observable. Then, no more oxygen was detected. Constant ponding of the filter surface was observed 1 day later, indicated by the weight of the column. The graph of the column mass (Fig. 3) shows that constant ponding occurred only 1 day after the oxygen concentration dropped to zero. McKinley and Siegrist (2010) underlined the fact that low oxygen concentrations near the inlet of the active zone were liable to create clogging through biological matter accumulation. Indeed, Nevo and Mitchell (1967) proved that the polysaccharide production did not decrease with oxygen concentration, whereas its consumption almost stopped. In other words, oxygen concentration was high enough for bacterial growth, but too low for mineralization of the biomass (McKinley and Siegrist, 2010). Moreover, physical clogging would have implied a modification of the hydraulic behavior of the column before the drop in oxygen concentration. This indicates that the clogging was caused by bacterial growth. From a more general point of view, oxygen concentration inside the filter can be determined by gas content measurements in field-scale vertical filters. Low values indicate that the filter is affected by clogging, at least around the measurement point.

Figure 4 shows water outflow over time. During Phase 2, the water outflow varied according to the arrival of the batches. It was higher at the beginning of the batch, and then decreased until the next batch. For the whole phase, the mean outflow was 4.8×10^{-3} l/min, with a standard deviation of 2.1×10^{-3} l/min. In Phase 3, the water outflow did not vary according to the batches, and the variation could not be linked to either the hydraulic or the pollutant load (SS or COD). The filter encountered constant ponding between batches. The depth of the ponding was not constant. For all of Phase 3, the mean outflow was 4.5×10^{-3} l/min with a standard deviation of 1.4×10^{-3} l/min. The mean water outflow value was lower than the one calculated during Phase 2, but was still sufficient to allow drainage of the filter. Outflow volume measurements have been used by Langergraber et al. (2003), among others, to monitor clogging. Concerning the present experiment, the outflow decreased 1 day after the absence of oxygen was determined inside the filter. Constant ponding between two batches (i.e. clogging) was observed simultaneously. Subsequently, the outflow velocity varied, while oxygen concentrations remained negligible in the filter. Although the filters started to clog, the outflow was sufficient to allow the infiltration of wastewater. This makes outflow volume observation unsuitable for early clogging prediction.

CONCLUSIONS

An experimental study was conducted to extend knowledge of the interplay between clogging and oxygen concentration trends in vertical flow filters for on-site wastewater treatment. This study focused on pilot-scale vertical filters. The second aim was to investigate available parameters allowing identifying early clogging in systems where it is not obviously observable, such as buried filters for on-site treatment. The study of the pilot-scale sand filters gave especial attention to the evolution of ammonium and nitrate concentrations in the system. The discussion dealt with ammonium transformation processes and led to the conclusion that nitrate outlet concentration monitoring is a means of predicting clogging, unlike ammonium concentration. The discussion then addressed oxygen transfer in the filter during normal operation as well as at the onset of clogging. Convective and diffusive fluxes were observed qualitatively during normal operation. The absence of oxygen was noted one day before constant ponding of the filters between two batches, which makes oxygen concentration an interesting indicator of incipient clogging in vertical flow filters. Finally, filter hydraulics were studied using the outlet water velocity, but proved to be insufficient for predicting early clogging problems in vertical flow filters.

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Figure 1. Pollutant loads for Part 1 (left), and Part 2 (right) of the experiment.

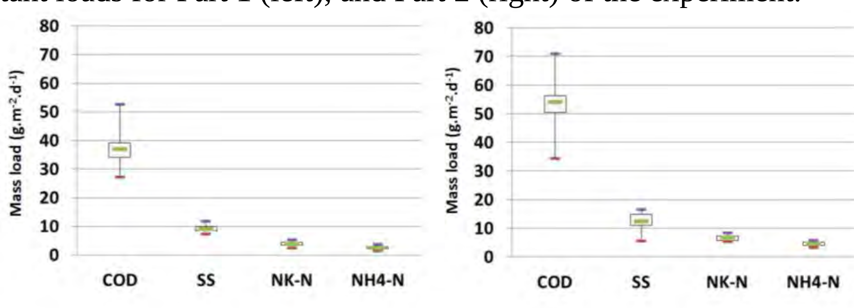


Figure 2. Mass balance for nitrogen forms in the filter.

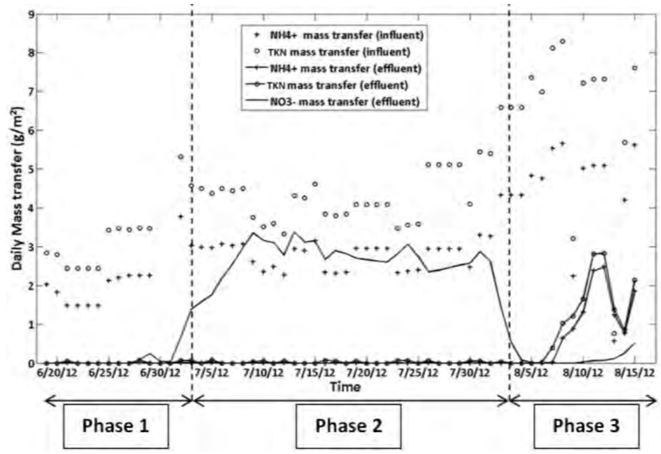


Figure 3. Mass of the column and oxygen concentration (% of $[O_2]_{sat}$) in the filter during Phase 2 and Phase 3.

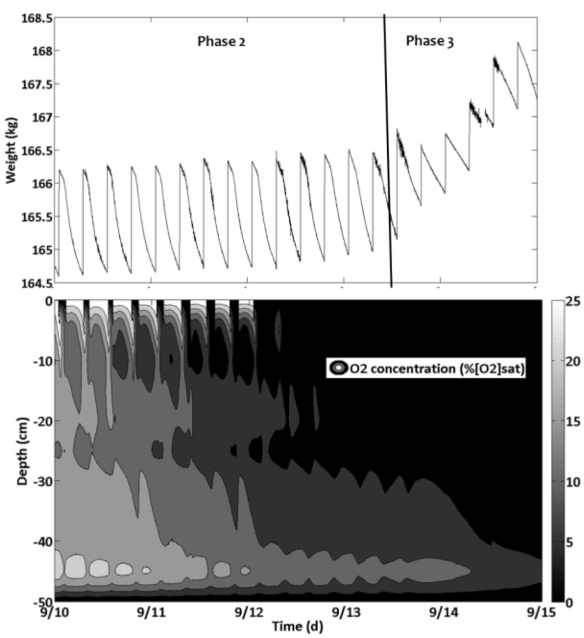
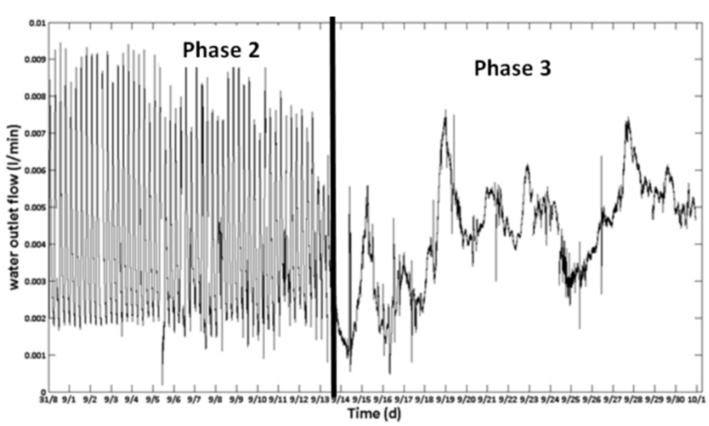


Figure 4. Water outflow during Phase 2 and Phase 3.



Treatment of Drip Dispersed Effluent in Imported Soils

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ABSTRACT

Soils and landscapes of the Table Rock Lake area around Branson, Missouri are thin, gravelly and steeply dissected while underlain by limestone with karst features. This area is susceptible to water from the surface and upper subsoil easily being transmitted into the ground water. This karst structure along with the dense development along Table Rock Lake is vulnerable to infiltration of contaminated waters from human activities (Casaletto and Borchelt, 2007). Previous research in the area pointed to onsite wastewater systems contributing to decreasing water quality (Casaletto and Borchelt, 2007). Monitoring was conducted to evaluate the performance of three renovated onsite systems that were included in the Table Rock Lake Wastewater Demonstration Project. The three systems were selected to demonstrate how advanced treatment technologies could be installed and operated in challenging site conditions in the Table Rock Lake area. These challenging conditions commonly exist of marginal soil-site receiving conditions and limited lot sizes. The sites included two seasonal resorts: Cape Fair Resort and the Lampe Resort, as well as the Shell Knob Restaurant South. These three systems included drip dispersal into imported soil after secondary treatment units.

MATERIALS AND METHODS

The Cape Fair Resort treatment system consisted of a septic tank, a BioMicrobics FAST® unit and drip dispersal into imported soil with a design flow of 7267 l/d. The Lampe Resort treatment system consisted of two septic tanks, followed by three Zabel SCAT® units operated in parallel, and drip dispersal into imported soil with a design flow of 5905 l/d. The Shell Knob South Restaurant treatment system consisted of a series of three septic tanks, a FAST® biological treatment unit and drip dispersal in imported soil with a design loading rate of 5678 l/d. Data were collected from each of the selected systems after installation to evaluate system conditions, treatment unit process performance and dispersal system performance. Monitoring was conducted at Lampe Resort, and Shell Knob South Restaurant from November of 2005 through July 2007. The Cape Fair Resort monitoring began when the onsite system was placed into operation in August 2006 and continued through July 2007. Unit process and water quality data were collected at representative sampling locations to evaluate the performance of the treatment systems and dispersal fields. Special attention was given to selection and placement of imported soil for the soil treatment and dispersal unit. The soil was specified to be less than 20 percent clay (silt loam, sandy loam, or loamy sand); harvested, transported, and placed at the site under dry conditions; and major roots and organic debris removed as well as any major coarse fragments. Generally, the imported soil was placed in 15 to 20 cm "lifts" to minimize compaction. Monthly sampling events were conducted for each of the three systems. Electronic rainfall gauges were installed at each site to provide information on how rainfall may affect subsurface sample concentrations. Sampling locations consisted of septic tank effluent, treatment system effluent and dispersal field effluent from subsurface sample collectors. Laboratory water quality measurements included 5-day biochemical oxygen demand (BOD_5), total suspended solids (TSS), ammonia nitrogen, nitrate nitrogen, total nitrogen, total phosphorus and fecal coliform using standard methods as outlined in Casaletto and Borchelt, 2007.

Subsurface sampling was conducted during each monthly site visit if sufficient sample volume was present in the sample containers. Monthly monitoring was conducted soon after rain events to assure the collection of fresh samples. Subsurface samples were collected using two methods. Lysimeter samples were collected in containers that remained inside of the riser portion of the monitoring device (Figure 1). The research team recognized the experimental nature of collecting subsurface water samples and the absence of standardized methods for water sample collection in subsurface drip dispersal fields (Sievers and Miles, 2001; Miles et al, 2007). Ideally, a properly operating drip dispersal field does not create saturated soil conditions and should not generate free water that can be collected in a gravity lysimeter (Hassan, et al 2005). Therefore, samples would be expected to be generated only during rain events. Sample volume and parameter concentrations could be affected by variables such as rainfall amount, frequency and intensity; the depth of soil from the drip tubing to the lysimeter; the location and number of drip emitters over the lysimeter; soil structure, and; temperature.

The data collected from subsurface samples were not intended to be compared to specific standards, but rather were intended to provide specific water quality information to better understand subsurface dynamics at each site monitored. Subsurface soil water collection devices were installed to collect and characterize subsurface water quality in the treated effluent dispersal fields at the three sites. Subsurface monitoring devices were also installed at each site in reference areas unaffected by the onsite system effluent to serve as experimental controls. Subsurface monitoring systems included gravity lysimeters. Innovative half-pipe and plastic sheet gravity lysimeters were designed and implemented to collect subsurface water samples for this project (Figure 1). Further information of the installation set-up and protocol are outlined in Casaletto and Borchelt, 2007. Both types of lysimeters were installed underneath drip lines at Cape Fair Resort and Shell Knob South Restaurant during the installation of the drip fields. Half-pipe lysimeters were installed into the Lampe Resort drip field several months after the drip field was installed. The drip tubing was cut and reconnected following the lysimeter installation. A plastic sheet lysimeter was not installed at Lampe due to the large area of the existing drip field that would need to be excavated and re-installed. The half-pipe lysimeter consisted of 30 cm PVC pipe that was cut in half lengthwise to create a 1.8 m long trough under the drip tubing (Figure 1). One of the most important variables in the drip field subsurface monitoring systems was the depth of soil between the drip tubing and the lysimeter. Site constraints such as depth to bedrock did not allow for standard depths between the drip tubing and the lysimeter. Soil depths ranged from 23 cm to less than 3 cm (Table 1).

Samples were generated from all lysimeters (Table 2). Lampe Resort generated the largest number of samples which was attributed to the minimal soil present between the drip tubing and the lysimeters. Very few samples were collected in the Shell Knob South Restaurant control lysimeters. The cause was not determined. The plastic sheet lysimeters for Cape Fair Resort and Shell Knob South Restaurant produced the greatest number of samples and were considered more effective than the half-pipe lysimeter.

RESULTS AND DISCUSSION

Loading rates at the Cape Fair Resort were proportional to room occupancy, which was greatest during the summer months and tapered to minimal occupancy during the off-season. All system loading rates were below the system design loading rate. Septic tank effluent BOD_5 and TSS concentrations were low (near or below 50 mg/L) during the off-season months. BOD_5 increased markedly during the busier summer season with concentrations ranging between 123 mg/L and 335 mg/L. TSS concentrations remained below 100 mg/L during the summer season, which indicated good settling conditions in the septic tank system. Effluent quality from the FAST system correlated with hydraulic loading rates. The smallest BOD_5 and TSS concentrations occurred during the off-season low-flow period and increased as flow increased during the summer months. BOD_5 and ammonia (not shown) concentrations in the plastic sheet lysimeter subsurface samples were near or below detection limits, which indicated consistent and thorough (80 to 100%) decreases in concentrations of these constituents as the treated effluent migrates through the soil column (Figure 2). Control lysimeter water sample concentrations were generally smaller than dispersal field samples. There were intermittent spikes in control datasets, such as fecal coliform concentrations in the Cape Fair Resort half-pipe lysimeter. All median concentrations in control datasets were at or below dispersal field concentrations (Table 3).

Subsurface monitoring datasets were reviewed for each of the monitored sites to determine the validity of the sampling methods. For Cape Fair Resort and Shell Knob South Restaurant, the plastic sheet lysimeter datasets were considered most representative of subsurface water quality. At Cape Fair Resort, the plastic sheet lysimeter produced sufficient samples for analysis of all parameters for five discrete samples, as compared to three with the half-pipe lysimeter. At Shell Knob South Restaurant, the half-pipe lysimeter samples potentially included potable water due to a water line break that occurred near the site. The plastic sheet lysimeter dataset was used exclusively for the Shell Knob South Restaurant dispersal field because of this water line break. Fecal coliform concentrations through the Cape Fair FAST system were reduced appreciably during the low-flow off-season months, but only marginal reduction was observed during the active summer months (Figure 3). As with all previous parameters, fecal coliform concentrations were consistently reduced in subsurface samples, ranging from less than detection limits to 1,530 colonies per 100 mLs (Figure 3).

As with the Cape Fair Resort, Lampe Resort loading rates were proportional to room occupancy which is greatest during the summer months and tapers to minimal occupancy during the off-season. Lampe Resort hydraulic loading rates during the 22-month monitoring period were typically at or less than 750 l/d during the off-season. In the first half of the study, septic effluent BOD_5 concentrations fluctuated widely, typically in the range from 20 to 300 mg/L. From July 2006, septic tank effluent BOD_5 concentrations generally remained below 50 mg/L. TSS concentrations were most often less than 50 mg/L in septic tank effluent samples, which indicated favorable settling conditions. SCAT effluent BOD_5 concentrations correlated with septic effluent measurements, with wide fluctuations in the first half of the study and more stable, smaller concentrations in the second half. BOD_5 measurements in the first half of the study fluctuated between concentrations of less than 10 mg/L BOD_5 to peak concentrations just below 100 mg/L. In the second half of the study, BOD_5 concentrations generally remained at or below 30 mg/L. TSS concentrations were typically below 20 mg/L, indicating the system achieved consistent and thorough suspended solids removal. The sporadic BOD_5 levels in summer 2006 in the treatment unit along with the small vertical separation of soil between the drip emitters and the lysimeter collection, could explain the small decrease in BOD_5 in the soil. Even though there was a minimal soil depth between the drip tubing and the half-pipe lysimeter the BOD_5 concentrations in subsurface samples collected in the half-pipe lysimeter were generally smaller than the Zabel SCAT effluent concentrations (Figure 4). Concentration reductions often exceeded 80 percent through the soil layer. Fecal coliform densities were reduced marginally by the SCAT system, with effluent concentrations generally above 10,000 colonies/100 mL (Figure 5). Subsurface fecal coliform densities were generally at or less than 1,000 colonies/100 mLs with concentrations below 100 colonies/100 mL in winter months.

The Shell Knob South Restaurant treatment system experienced foaming problems on start-up in the FAST unit vent pipe. The new owners of the restaurant opted to discontinue blower operation for the remainder of the study period. The peak hydraulic loading rate of 4164 l/d occurred in April 2006. Restaurant ownership changed in 2006 and hours of operation were reduced as were treatment plant loading rates. Business activity increased in the spring and summer of 2007 as reflected by increasing hydraulic loading rates, which approached 800 gallons per day in July 2007. FAST system BOD_5 concentrations were generally less than 100 mg/L with only one measurement greater than 150 mg/L (350 mg/L in June 2006). TSS concentrations were generally less than 50 mg/L. BOD_5 percent concentration reductions through the FAST system were typically between 70 and 95%, which indicated the system was capable of assimilating most of the restaurant organic load. BOD_5 and ammonia (not shown) concentrations in the plastic sheet lysimeter subsurface samples were consistently smaller than pump tank effluent samples (Figure 6). BOD_5 concentrations were generally below 20 mg/L with concentration reductions consistently greater than 80% in the soil. Winter and spring ammonia concentrations were typically below 1 mg/L but summer and fall concentrations increased to between 3 and 5.5 mg/L. Fecal coliform reductions through the FAST system were consistent but marginal, with typically less than one log reduction observed

(Figure 7). Subsurface concentrations were also smaller, but to varying extents. Greater than two log reductions were observed for several events.

SUMMARY

Subsurface water median concentrations using imported soil with drip dispersal for all parameters were lowest at Cape Fair Resort (Table 4). BOD_5 and ammonia concentrations were frequently at or below method detection limits. Median total phosphorus and fecal coliform concentrations were at or below the water quality criteria for surface discharging mechanical treatment plants in the Table Rock Lake watershed. Wide ranges in all parameters were observed in the Lampe Resort and Shell Knob South Restaurant subsurface samples. However, median BOD_5, ammonia and fecal coliform concentrations in soils for these two sites were all less than typical surface water discharge effluent limits for a disinfected effluent.

Effluent BOD_5 and TSS concentrations from the advanced systems were consistently below 20 mg/L indicating thorough and reliable organic and solids removal typical of surface discharging mechanical treatment systems. Plastic sheet and half-pipe gravity lysimeters are effective in collecting subsurface samples. Dispersal field subsurface sample concentrations were consistently smaller for all water quality parameters measured. For all three systems with gravity lysimeters, median BOD_5, ammonia and fecal coliform concentrations in the soil were below effluent limitations typically issued to mechanical surface discharging systems with nitrification and disinfection unit processes. Subsurface sample concentrations for fecal coliform indicated little, if any, correlation to rainfall amounts.

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Table 1. Soil Depth and Drip Tubing Placement for each Site.

Field Descriptor	Cape Fair Resort	Lampe Resort	Shell Knob Restaurant South
Soil Depth above drip tubing	30 cm	30 cm	30 cm
Soil depth between drip line and lysimeters	13 cm	< 3 cm	23 cm
Drip tubing orientation over lysimeters	parallel	perpendicular	parallel

Table 2. Number of Subsurface Effluent Samples Collected at Each Site.

Monitoring Site	Dispersal Field	Control
Cape Fair Resort		
Plastic Sheet Lysimeter	5	9
Half-Pipe Lysimeter	3	7
Lampe Resort		
Half-Pipe Lysimeter	14	9
Shell Knob Restaurant S.		
Plastic Sheet Lysimeter	18	1
Half-Pipe Lysimeter	14	2

Table 3. Median sample concentrations for BOD5, Ammonia, Phosphorus, and Fecal Coliform in Soil Dispersal Monitoring Sites and Control Sites.

	Cape Fair Resort	**Lampe Resort**	**Shell Knob Restaurant S.**
	BOD5 (mg/L)		
Plastic sheet lysimeter			
Dispersal Field	3	----	3
Control	3	----	3
Half-pipe lysimeter			
Dispersal Field	11	3	4
Control	3	3	7
	Ammonia (mg/L)		
Plastic sheet lysimeter			
Dispersal Field	0.02	----	0.02
Control	0.02	----	1.28
Half-pipe lysimeter			
Dispersal Field	0.03	0.62	0.61
Control	0.02	0.14	0.44
	Phosphorus (mg/L)		
Plastic sheet lysimeter			
Dispersal Field	0.46	----	0.06
Control	0.14	----	0.25
Half-pipe lysimeter			
Dispersal Field	0.98	1.17	1.10
Control	0.15	0.19	0.16
	Fecal Coliform (colonies/100ml))		
Plastic sheet lysimeter			
Dispersal Field	81	----	23
Control	63	----	5
Half-pipe lysimeter			
Dispersal Field	45	186	153
Control	99	5	18

Table 4. Median BOD5, Ammonia, Phosphorus, and Fecal Coliforms for Septic Tank Effluent, Treatment System, and Soil Dispersal Field for reported systems

	Cape Fair Resort	Lampe Resort	Shell Knob Restaurant South
BOD5 (mg/L)			
Septic Tank Effluent	108	36	343
Treatment System Effluent	12	17	59
Subsurface Samples	3	3	4
TSS (mg/L)			
Septic Tank Effluent	46	29	64
Treatment System Effluent	12	7.8	32
Ammonia (mg/L)			
Septic Tank Effluent	6.1	5.6	5.3
Treatment System Effluent	4.8	5.2	5.7
Subsurface Samples	0.02	0.62	0.61
Phosphorus (mg/L)			
Septic Tank Effluent	2.6	3.0	3.5
Treatment System Effluent	2.1	2.8	3.0
Subsurface Samples	0.5	1.2	1.1
Fecal Coliform (colonies/100 mL)			
Septic Tank Effluent	551,000	103,000	160,000
Treatment System Effluent	12,060	8,290	50,000
Subsurface Samples	81	186	153

Figure 1. Diagram of Half Pipe Lysimeter used in study.

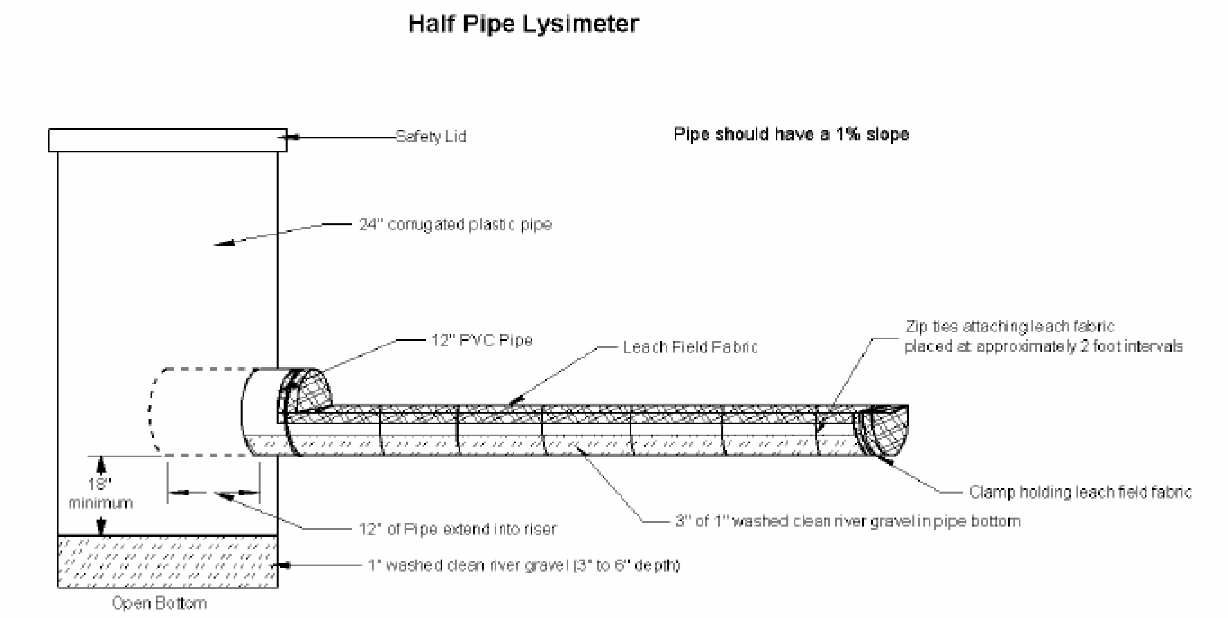


Figure 2. BOD_5 Concentrations and Percent Decrease from FAST Treatment Unit to Soil Dispersal Field Cape Fair Resort.

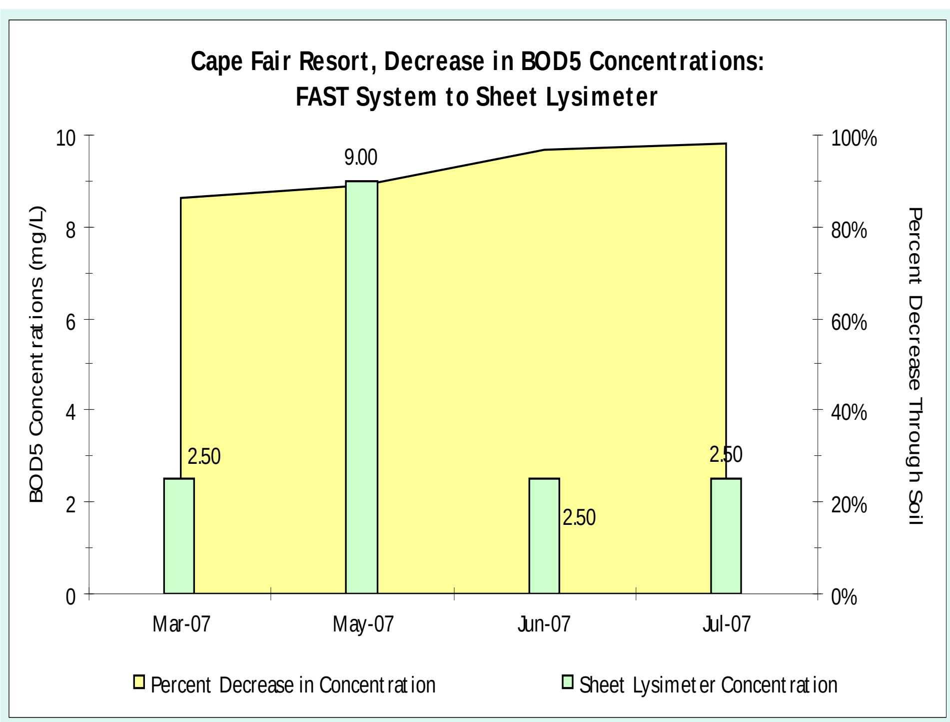


Figure 3. Fecal Coliform Numbers in Septic Tank Effluent, FAST Treatment System and Soil Dispersal Field over time for Cape Fair Resort.

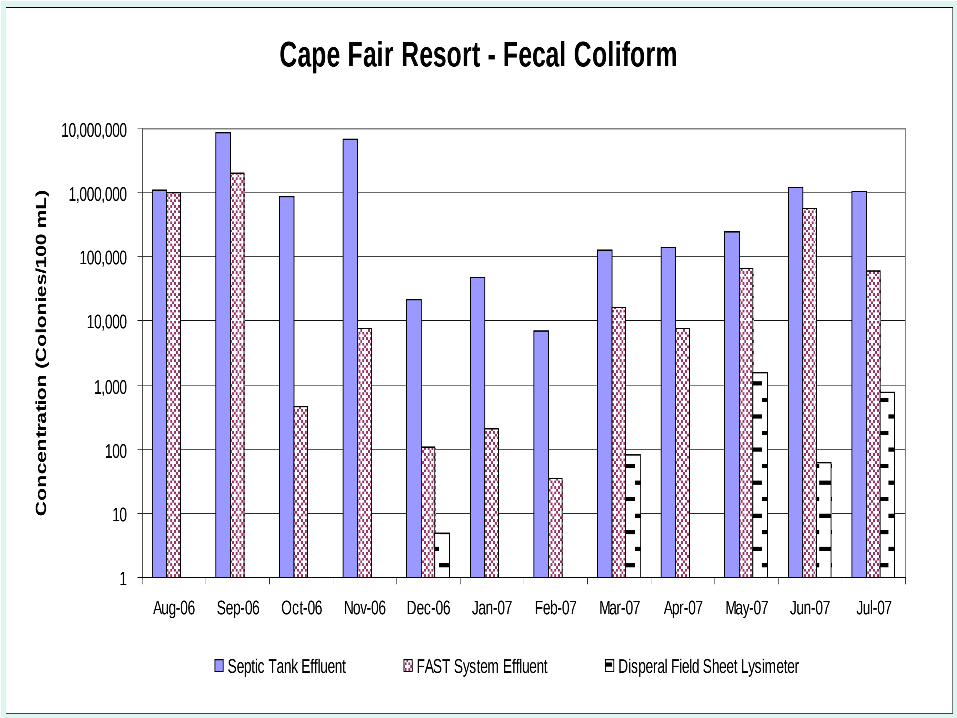


Figure 4. BOD_5 Concentrations and Percent Decrease from Zabel SCAT Treatment Unit to Soil Dispersal Field Lampe Resort.

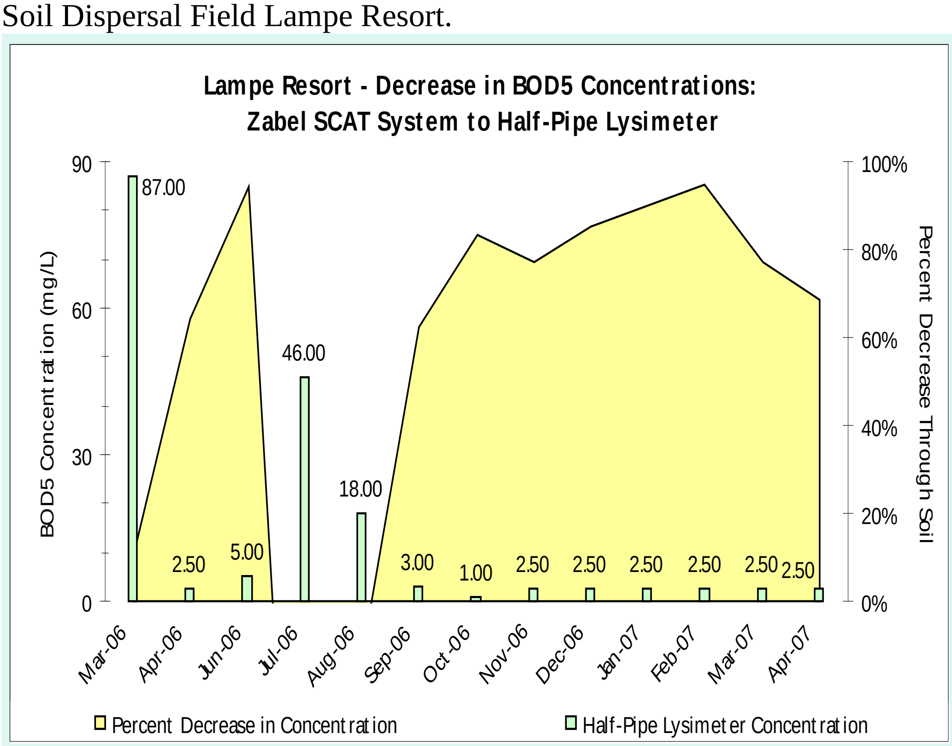


Figure 5. Fecal Coliform Numbers in Septic Tank Effluent, Zabel SCAT Treatment System, and Soil Dispersal Field over time for Lampe Resort.

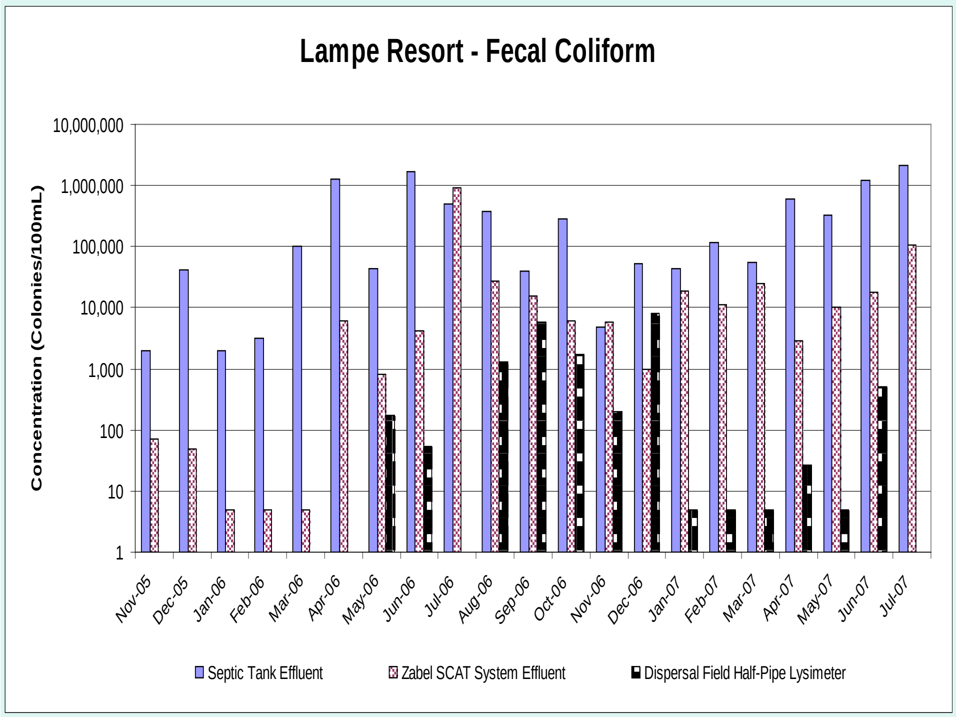


Figure 6. BOD_5 Concentrations and Percent Decrease from FAST Treatment Unit to Soil Dispersal Field Shell Knob Restaurant South.

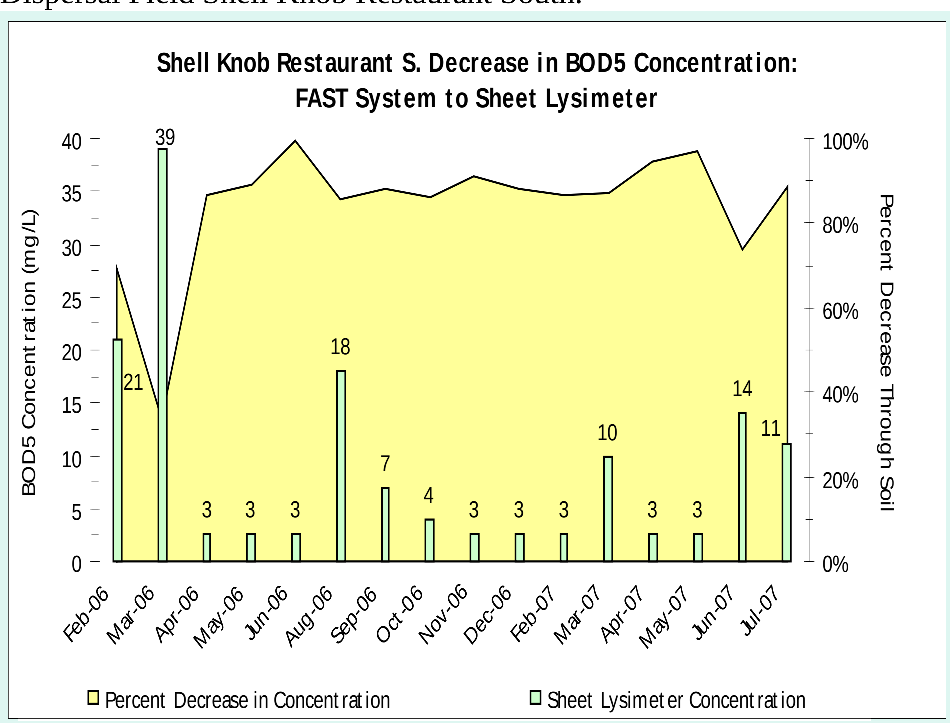
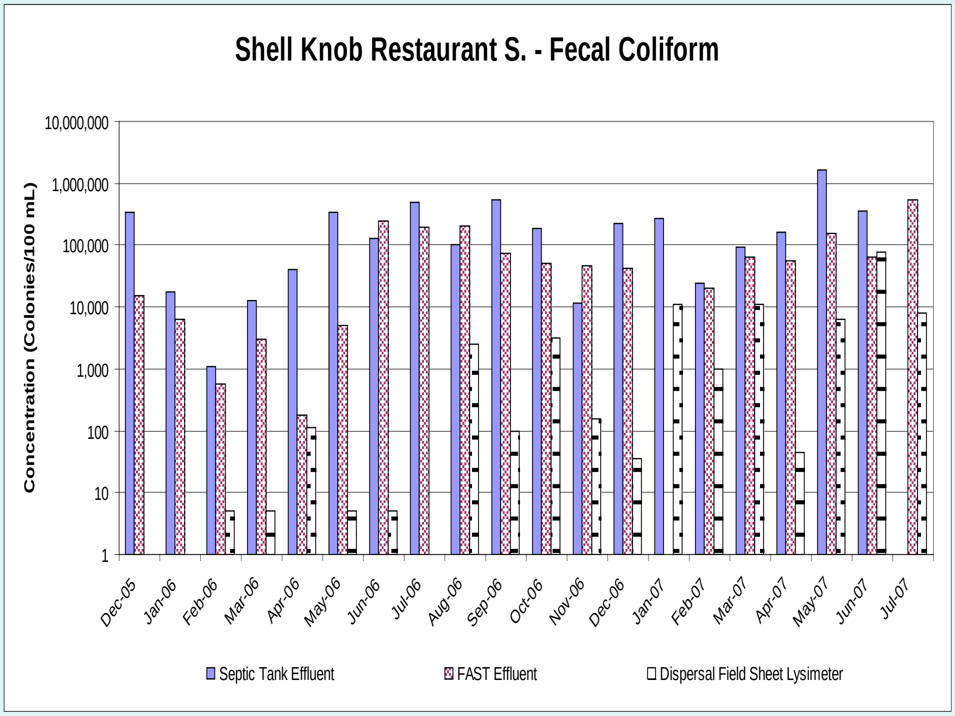


Figure 7. Fecal Coliform Numbers in Septic Tank Effluent, FAST Treatment System and Soil Dispersal Field over time for Shell Knob Restaurant South.



Hydrologic Assessment for Wastewater Land Disposal

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ABSTRACT

Proper field assessment of the soils hydrological properties is the most important aspect of the soil/site assessment and design for wastewater disposal/dispersal. Whether for a large or small system, wastewater application at a given land site depends on a number of factors including natural precipitation, surface flow, and subsurface flow. For this reason, a hydrological assessment of the ability of the soil and the site to assimilate the added water must be performed. For small systems (e.g., single family housing) the hydrological assessment is performed indirectly by determining the appropriate long term acceptance rate (LTAR) through morphological characterization, or by in-situ measurements of the soils percolation rate. For large systems, a more elaborate assessment of the soil hydrologic properties, coupled with modeling of the water flow from the system/site would be necessary to perform. For most practical applications, the hydraulic properties are limited to the thickness of the vadose zone (depth to the water table), thickness of the aquifer, and the hydraulic conductivity within the vadose zone and the aquifer. Due to the difficulties associated with determining the unsaturated hydraulic conductivity curve at various depths and locations within the vadose zone, the measured saturated hydraulic conductivity (K_{sat}) is used for modeling purposes. Although the proper procedures for determining K_{sat} for both the saturated and unsaturated zones are readily available, some consultants and regulatory officials fail to follow the proper procedure and methodology for the specific site being evaluated. This paper addresses the proper procedures for determining the K_{sat} of the vadose zone and the saturated zone utilizing the constant head well permeameter method and the slug test, respectively. Specifically, field data collection and the use of an appropriate model for determining K_{sat} will be discussed.

Despite the wide spread use of onsite wastewater disposal systems commonly known as septic systems (with the exception of the general publication by the USEPA, 2002), there are no national standards for their design, installation, operation, and management. Instead, the laws and regulations governing onsite systems are generally administered by states and in some cases local governments. As a result, the criteria used for soil/site evaluation and design of onsite wastewater dispersal systems vary considerably across the United States. In general, onsite systems are designed to handle a given volume of wastewater. The long-term acceptance rate or LTAR refers to the volume of wastewater that can be safely applied daily to a unit dispersal area (known as the drainfield) without causing a hydraulic failure. Many states, including North Carolina, rely on morphological characterization of the soil for conducting soil/site evaluation and determining the type and the appropriate LTAR for onsite subsurface systems (NCDHHS, 2014). Other states may use the percolation test as the basis for soil/site evaluation and determination of LTAR. Since installing an onsite system can significantly impact the local hydrology of the drainfield area, the design of large onsite dispersal systems requires a more elaborate soil/site assessment and perhaps modeling of water flow from the dispersal area of the respective system.

In a properly functioning onsite system, wastewater applied below the soil surface through trenches or drip irrigation lines must infiltrate the soil and move vertically down through an unsaturated zone before reaching a water table or an impermeable layer (Amoozegar et al, 2005; Amoozegar et al, 2008). Once in the groundwater or in a saturated zone above the impermeable layer, water must move laterally away from the drainfield area. The designer of the system, and the regulatory official approving the system, must make sure that the site under consideration has the capacity to assimilate the added wastewater by conducting a hydrological assessment. Assessment of water flow from the trenches in the vadose zone (Radcliff et al., 2005; Beach and McCray, 2003; White and West, 2003; Huntzinger et al., 2001) and evaluation of lateral flow and/or groundwater mounding within the aquifer (Amoozegar and Martin, 1977; Poeter et al., 2005, Korkmaz, 2013) require knowledge of the hydraulic properties of both saturated and unsaturated zones. For the saturated zone, the main parameters of interest are saturated hydraulic conductivity (K_{sat}), thickness of aquifer, and specific yield (or drainable porosity). For the vadose zone, the parameters of interest include unsaturated hydraulic conductivity (K_{unsat}), thickness of the unsaturated zone, and soil water characteristics. Since K_{unsat} depends on soil water pressure head or water content, obtaining a complete unsaturated hydraulic conductivity curve will be impractical for most applications. As an alternative, a mathematical model, using soil water pressure head and K_{sat} of the unsaturated zone, can be used to estimate K_{unsat} for modeling purposes (Gardner, 1958; van Genuchten, 1980).

There are a number of procedures for in situ determination of soil saturated hydraulic conductivity above a water table (in the vadose zone) and below a water table (within an aquifer) (Amoozegar and Wilson, 1999; Reynolds and Elrick, 2002). Although capillary fringe plays an important role in lateral movement of water and transport of solutes (Silliman, 2002; Abit et al., 2008), due to its nature (i.e., almost saturated but under tension), no in-situ technique is available for measuring its hydraulic conductivity. Therefore, our discussion will only address the unsaturated zone above the capillary fringe and the saturated zone below it. Also, it should be noted that due to the differences in the modeling approaches for determining K_{sat}, as well as the volume of soil that impact field data, none of the techniques available for in situ measurement of K_{sat} in the saturated or vadose zone provides a unique value for it. This is mainly because the models for calculating K_{sat} are generally based on assumptions that may not hold true under most natural field conditions. For example, most models assume the soil to be homogeneous and isotropic with water flow happening rather uniformly in certain fashion. When selecting a procedure, one must consider the practical application of the results. In some cases, it is more practical to select a model that makes simplified assumptions resulting in a less elaborate and costly procedure in place of a more time consuming and complicated procedure that uses a more sophisticated model based on similarly unrealistic assumptions. However, most commonly used procedures published by ASTM or Soil Science Society of America are standardized and can be easily replicated. The main objective of this paper is to present an overview of the proper procedures for the in situ determination of the saturated hydraulic conductivity of both the saturated zone below a water table (i.e., within an unconfined aquifer) and the unsaturated zone above the capillary fringe by the slug test and the constant head well permeameter method, respectively.

Methods for Determination of K_{sat} Below a Water Table

The available in-situ methods for determination of K_{sat} can only be used in areas where the saturated zone extends horizontally in all directions. This means that the water table at the site of measurement must be horizontal. For small pockets of saturated soil (e.g., perched water table above an impermeable lens) K_{sat} can only be determined in a laboratory using intact core samples from the saturated soil volume. Also, unless water from an external source is applied to the groundwater, the groundwater itself is used for measurements of K_{sat} below a water table. Therefore, there is no need to transfer water to remote locations, make corrections with respect to temperature, or be concerned about the chemical quality of the water to impact water flow through changes in soil structure.

Skaggs (1976) introduced a methodology for determining the saturated hydraulic conductivity-drainable porosity ratio using a single drainage ditch or between two parallel drainage ditches. Knowing the drainable porosity, K_{sat} can then be easily calculated. Most other techniques use a hole (e.g., well) to determine K_{sat} and perhaps thickness of the aquifer. In some applications, such as installation of deep drinking water wells, one or more relatively deep wells may be required for 24- or 48-h pump tests (Freeze and Cherry, 1979). For other applications, such as designing drainage systems for managing water table in an agricultural field or under a wastewater dispersal system, our main concern may be water flow within the upper part of an unconfined aquifer. For these cases, we can employ a method that may only require hand tools for boring a shallow well. The most common method for determining K_{sat} within the upper part of the saturated zone using hand tools is the auger-hole method (Boast and Kirkham, 1971). Other procedures that are available for determining K_{sat} include piezometer method, two well and four-well methods (Amoozegar and Wilson, 1999). The most versatile procedure, which allows measurement near the water table or at deep depth is slug test (Bouwer and Rice, 1976). The slug test, in principle, is similar to the piezometer method presented by Kirkham (1945), and later modified by Frevert and Kirkham (1948) and Luthin and Kirkham (1949).

Slug Test

When utilizing the slug test a cylindrical hole of known diameter ($2r_w$) is dug to the desired depth below the water table (H), and a well casing of internal diameter $2r_c$, attached at the bottom to a section of perforated pipe of the same diameter with high conductivity, is inserted into the hole (Fig. 1). To maintain the integrity of the well, the space between the well walls and the perforated section of the casing is packed with a high conductivity sand or gravel. The length of the packing material in the well (L) must be equal or greater than the length of perforated section of the casing, and the conductivity of the casing must be great than the conductivity of the packing materials. [NOTE: Local or state regulations governing well construction may require the packing material to cover a certain distance above the perforated section of the casing.] After placing the packing materials around the casing, the space between the solid section of the casing and the well walls is sealed by backfilling with bentonite. Finally, the thickness of the aquifer , D (i.e., the distance between the impermeable layer and the water table) must be determined independently at the site or be estimated from the available data.

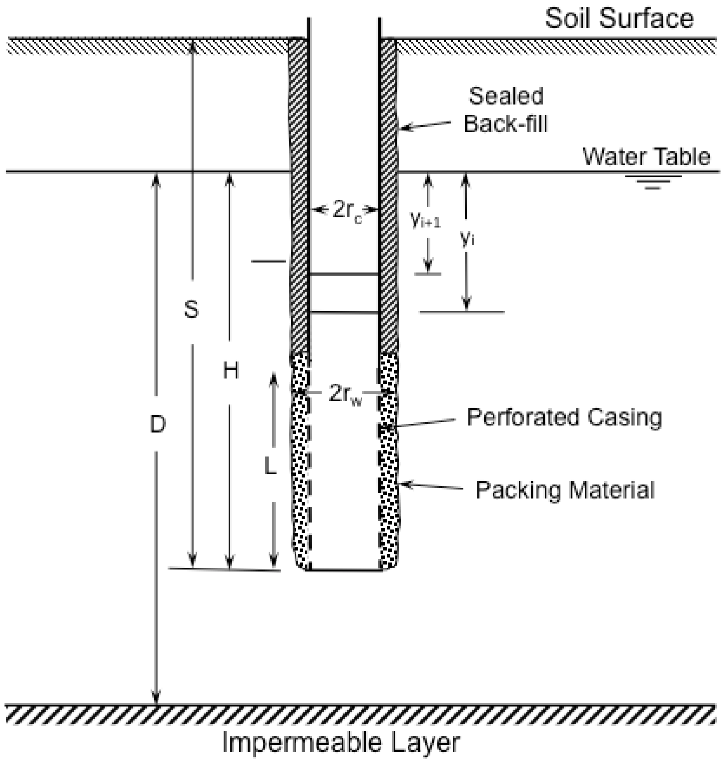

Figure 1. Schematic diagram of the slug test for partially perforated well in an unconfined aquifer (adapted from Bower and Rice, 1976).

Filed Data Collection:

Prior to conducting the slug test, water is removed from the well and the level of the water is then allowed to return to near its static level in the well several times. This is similar to developing a newly constructed well. To conduct the slug test, the level of water in the well is allowed to reach its static level. Then, the level of water in the well is lowered very quickly a distance *y* (see Fig. 1) by fast removing a volume of water, referred to as a slug of water, from the well. As an alternative, a solid cylinder (with a diameter of approximately 1-2 cm less than the inner diameter of the casing in the well) is lowered to below the static water level in the hole. After allowing the water level to reach the static level, the solid cylinder is removed from the well at once, causing the water level in the well to drop very quickly. By a rapid water level drop, we can assume that there is no drawdown around the well while the water level rises in the well. The difference between the water level in the well and the water level outside the well creates a hydraulic gradient forcing water from the aquifer to enter the well and raise the water level in the casing. Immediately after lowering the water level, the level of the water in the well is measured with time to determine its rate of rise. The rate of rise of water in the well, along with other parameters that will be discussed later, is then used to calculate the K_{sat} of the aquifer. We refer to this type of slug test as “the rising head slug test.” The slug test can also be performed by raising the water level in the casing quickly and allowing it to fall. Instead of applying water to the well after it reaches its static level, the solid cylinder is quickly inserted to below the static water level and the level of water in the well is measured with time. We refer to this type of slug test as “the falling head slug test.” It should be noted that ‘falling’ and ‘rising’ slug tests can be performed alternately by inserting the solid cylinder in the hole for raising the water level, and then removing it for lowering the water level very quickly.

Calculation:

Using the field data, saturated hydraulic conductivity is calculated by

$$K_{sat} = \frac{r_c^2 \ln(R_e / r_w)}{2L} \times \frac{\ln(y_o / y_t)}{t} \qquad [1]$$

where y_0 is the distance between the water level in the hole and the static water level (y) at time zero, y_t is the y at time t, R_e is a shape factor that must be determined based on the geometry of the hole, and the other parameters are as defined above. Using an electrical resistance analog, Bower and Rice (1976) estimated the R_e values and expressed the dimensionless parameter $\ln(R_e/r_w)$ by

$$\ln(R_e / r_w) = \{\frac{1.1}{\ln(H / r_w)} + \frac{A + B\ln[(D-H)/r_w]}{L / r_w}\}^{-1}, \text{ when } D > H \qquad [2]$$

and

$$\ln(R_e / r_w) = [\frac{1.1}{\ln(H / r_w)} + \frac{C}{L / r_w}]^{-1}, \text{ when } D = H \qquad [3]$$

where A, B, and C are obtained for various L_e/r_w values from a set of graph presented by Bower and Rice (1976). As an alternative to using the above models, a commercially available program, Super Slug (Starpoint Software Inc., Mason, OH), can be conveniently used to determine K_{sat}.

Bower and Rice (1976) have set a limit of 6 for the value of $\ln[(D - H)/r_w]$ in Equation [2]. They have indicated that a value of 6 should be used when $\ln[(D - H)/r_w] > 6$. Also, care should be taken to select a linear section of the curve relating the rise or fall of the water level in the well and time when determining the K_{sat} manually using Equation [1] or a program such as Super Slug.

Comments:

The slug test can be used to measure K_{sat} of the upper part of an aquifer using a hand-bored auger hole as well as at deeper depths using a drilling rig (without using drilling mud) for constructing the well. The lengths of the perforated casing and the gravel packed zone at the bottom of the well can be adjusted for measuring the K_{sat} of individual soil layers in layered aquifers. For measurements in high conductivity aquifers, the water level in the test well may fall or rise very rapidly. For these situations, a small pressure transducer, capable of measuring the depth of water to within 1 cm, can be placed at a fixed location at the bottom of the well below the level where the bottom of the solid cylinder will be when fully immersed below the static water level. Care should be taken to allow the water level in the well to reach static level prior to performing falling or rising slug tests. Theoretically, any diameter well can be used for measuring K_{sat} by this technique. According to Bower (1989), small diameter wells (e.g., 5 cm) may not yield reliable results for the zone under consideration. For most practical applications, well diameters ($2r_w$) ranging from 10 cm (4 in) with a 5 to 7.5 cm (2 to 3 in) casing to 25 cm (10 in) diameter with a 10 to 15 cm (4 to 6 in) casing can be used conveniently.

Methods for Determination of K_{sat} Above a Water Table

To measure soil hydraulic conductivity in the vadose zone water must be applied to the soil. As a result, the chemical composition of the water and its temperature may have a substantial impact on the measured K_{sat} value. Therefore, every attempt should be made to use water with a similar composition as soil water in the area under evaluation. Also, temperature has a profound impact on the viscosity of water, therefore, the temperature of water that infiltrates the soil must be considered when determining the K_{sat} value.

Constant Head Well Permeameter Method

There are a number of procedures for measuring K_{sat} of the unsaturated zone (Amoozegar and Wilson, 1999; Reynolds and Elrick, 2002). Some of the techniques, such as the double cylinder infiltrometer or the air-entry permeameter are difficult to perform at deep depth because of the requirement for excavation of a large pit. Perhaps the most convenient, and least labor-intensive procedure for determining K_{sat} of the unsaturated zone is the constant head well permeameter method, also known as shallow well pump-in and borehole permeameter method. In this technique, K_{sat} is measured by applying water to a cylindrical auger hole bored to the desired depth, maintaining a constant depth of water at the bottom, and measuring the rate of water flow into the hole after reaching steady-state condition. Originally, this procedure was developed more than 60 years ago (Zangar, 1953), which required the excavation of a large diameter hole (in the order of 10 cm or more), and took a relatively long time (in the order of 24 hours) to reach a steady-state condition. As a result, a large volume of water was required for conducting the test. Theoretical evaluation of water flow from a cylindrical auger hole and field measurements of K_{sat} by this procedure showed that for most practical applications, measurements can be made in a small diameter hole (e.g., 6 cm) in a few hours (Reynolds et al., 1983; Talsma, 1970; Talsma and Hallam, 1980).

Currently, there are a number of commercially available devices for measuring K_{sat} using this procedure. These commercial devices include, but may not be limited to the Aardvark Permeameter (Soilmositure Corp., Santa Barbara, CA), Compact Constant Head Permeameter (Ksat, Inc., Raleigh, NC), Guelph Permeameter (Soilmositure Corp), Johnson Permeameter (Johnson Permeameter, LLC, Saluda, NC), and Perm-It Permeameter (American Manufacturing Company, Inc., Manassas, VA). Other devices using a float (Luthin, 1978; Stephen et al., 1987) or a home-made Mariotte bottle systems (Talsma and Hallam, 1980) can be used for applying water to the cylindrical hole under a constant head. Theoretically, there is no limit to the depth where K_{sat} can be measured, but the limitations are availability of equipment for boring a cylindrical home of know diameter, maintaining a constant depth of water at the bottom of the hole, and measuring the rate of water flow into the hole after reaching steady-state condition.

Field Data Collection:

In this procedure an auger hole of radius *r* is dug to the desired depth. Although auger holes in the range of 4 to 12 cm in diameter can be used, a 6-cm diameter hole is highly recommended to allow measurements using a few liters of water. The soil profile can be described during the excavation of the hole. Using a planer auger of the same diameter as the auger, the bottom of the hole is shaved to form a cylindrical-shaped cavity. If needed, a round nylon brush or similar devices (see Reynolds and Elrick, 2002) is then used to scrape off the hole side-wall and remove any smearing that the auger may cause at the bottom section of the hole. Various size auger sets for constructing the hole are available commercially. It should be noted that the diameter of the bottom section of the auger hole used for measuring K_{sat} must be known, and that the diameter of an auger hole constructed using a hand auger is generally larger than the nominal size of that auger. If the auger manufacturer specification does not specify the diameter of the hole that can be dug with the auger, the user must determine the hole diameter within 1 to 2 mm accuracy.

After determining the depth of the hole, water is applied to the auger hole to the desired depth (referred to as head *H*) using an appropriate apparatus, such as the one shown in Fig. 2. [NOTE: Although the published procedures and commercial permeameter manuals generally do not specify an exact value for *H*, there are some general limitations for the depth of the water that must be maintained for determining the steady-state rate of water flow. Amoozegar and Wilson (1999), for example, require the H/r ratio for determining K_{sat} using the Glover model to be ≥ 5.] The depth of water in the hole must be measured a few times to assure that a constant head of water is maintained. After establishing the desired constant depth of water, time is allowed for the water to infiltrate the soil until reaching steady-state condition. Theoretically, steady-state condition for measuring K_{sat} can be defined as the time during which the rate of water flow from the hole, where measurement is conducted under a constant depth of water, reaches a constant value (i.e., no longer changes with time). After allowing pre-saturation of the soil around the hole, the flow rate should reach a quasi-steady-state condition during which the flow rate moves up and down slightly (within a few percentage) around an average value. To determine this average value, it is best to plot the rate of water flow (or the calculated K_{sat} values) versus time and pass a smooth curve through them using a manually (referred to as fitting a curve by eye) or a mathematically (e.g., statistically) best-fitted curve. The steady-state flow rate is reached if the tail end of this fitted curve is nearly horizontal without showing an upward or downward trend. The average flow rate for the last three to five measurements after reaching steady-state can be used for calculating K_{sat}.

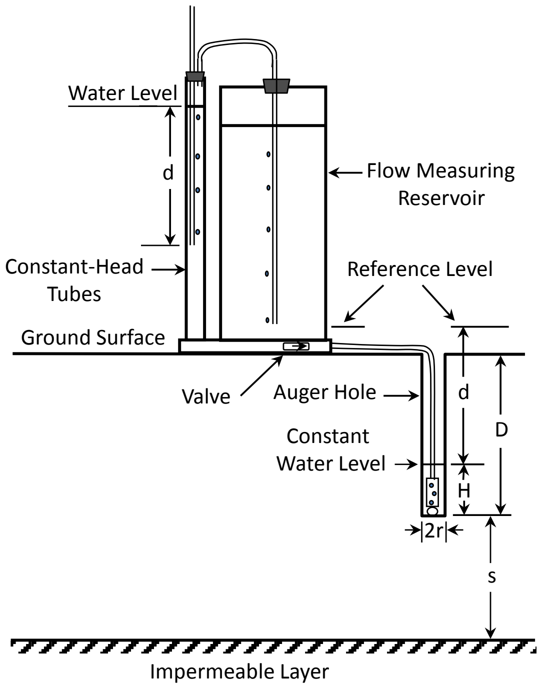

Figure 2. Schematic diagram of a constant head permeameter for maintaining a constant depth of water (H) at the bottom of a cylindrical auger hole of radius r and depth D.

Calculation:

There are a number of models for calculating K_{sat} using field data. The general equation for calculating K_{sat} by these models is

$$K_{sat} = AQ \qquad [4]$$

where Q is the steady-state flow rate from the hole into the soil and A is a factor that must be calculated with a model. In the Glover model, developed based on only the saturated flow of water around the hole, the A factor is given by

$$\mathrm{A} = [\sinh^{-1}(H/r) - (1 + r^2/H^2)1/2 + (r/H)]/(2\pi \mathrm{H}^2) \qquad [5]$$

where $\sinh^{-1}$ is the inverse hyperbolic sine function and r and H are as defined before (Zangar, 1953). Stephens and Neuman (1982) presented regression equations for calculating K_{sat}. Reynolds et al. (1983) and Philip (1985) used a set of similar assumptions regarding saturated and unsaturated flow around the auger hole and developed models that contain K_{sat} and a parameter related to the capillary properties of the soil, referred to as sorptive number α. They used the empirically developed equation relating the unsaturated hydraulic conductivity $K(h)$ to K_{sat} by

$$K(h) = K_{sat}\exp(\alpha h) \qquad [6]$$

where h (unit of length, L) is the soil water pressure head, h_i is the initial soil water pressure head, and α (L^{-1}) is a constant. Reynolds et al. (1985) (also see Elrick et al., 1989) presented an analytical model for calculating for A

$$A = C/(2\pi H^2 + \pi r^2 C + 2\pi H/\alpha) \qquad [7]$$

where C is related to r and H and is obtained from graphs developed for different soils (see Elrick et al., 1989) or from fitting equations presented by Radcliffe and West (2000). The model presented by Philip (1985) also contains the two unknown parameters K_{sat} and α but will not be discussed here. In order to determine K_{sat} and the parameter related to the unsaturated flow, Reynolds et al. (1985) offered the simultaneous equation approach, which requires two sets of field measurements of the steady-state flow rate from a cylindrical auger hole under two different heads (i.e., depth of water in the hole). However, Amoozegar (1989) and Salverda and Dane (1993) showed that the simultaneous equation approach may result in negative K_{sat} values. To overcome the negative values that could result for both K_{sat} and the unsaturated flow parameter, Elrick et al. (1989) presented a new approach for calculating K_{sat} using pre-assigned values to α based on soil texture and structure.

Comments:

The constant head well permeameter method is perhaps the most convenient field procedure for measuring K_{sat} at different depths. In order to obtain reliable field data, the procedures regarding the construction of the cylindrical auger hole, knowing the diameter of the hole and the constant depth of water at the bottom of the hole within 1 to 2 mm, and measuring the flow rate after reaching steady-state flow rate must be followed precisely. Also, an appropriate model must be used to calculate K_{sat} using field-collected data. As indicated previously, the simultaneous equation approach, introduced by Reynolds et al. (1985) and suggested for use by Reynolds and Elrick (2002) should be considered unreliable because it may produce negative values for K_{sat} or the unsaturated flow parameter. The Glover model (Eq. [5]), on the other hand, is simple, requiring measurements of Q under only one H, and does not require an estimation of any soil parameter. The Glover model has been criticized because only the saturated flow around the hole was considered in its development (Reynolds et al., 1983, 1985; Radcliffe and West, 2000). Amoozegar (1989) demonstrated that the Glover equation results are relatively close to the results obtained by Philip (1985) and Reynolds et al. (1985) models for most practical applications. In addition, a number of assumptions were used in the development of the latter two models: the soil under consideration is homogeneous and isotropic, a saturated bulb forms around the auger hole at steady-state, and that the unsaturated hydraulic conductivity is related to K_{sat} and soil water pressure head by the empirically derived Eq. [6]. Considering these assumptions, and the closeness of the calculated K_{sat} values by the available models for similar conditions, the criticism of the Glover equation seems to be unjustified. The individual in charge of measuring K_{sat} in situ should have enough understanding of the procedure and the respective model for calculating its value to produce reliable results.

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Indicators of Soil Quality in a Wastewater-Amended Semi-Arid Soil

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ABSTRACT

Forty-two surface soil samples (0-0.15 m) were collected from a field located in Aztec, NM that had been cleared of native vegetation, sown with a pasture mix, and irrigated with reverse osmosis reject wastewater (electrical conductivity (EC) of 2.73 dS/m) for two consecutive years, and analyzed for multiple soil quality indicators (SQI). This was done as part of a preliminary study with the objective to investigate the relationships among the measured SQI through multivariate statistics, in order to detect data structures and reduce the number of measurements needed for soil quality assessment. Potential SQI measured included mean weight diameter of dry aggregates, dry aggregates > 2 mm (D > 2 mm), dry aggregates < 0.25 mm (D < 0.25 mm), wet aggregate stability, permanganate-oxidizable carbon, soil organic carbon, EC, pH, sand, silt and clay content, and chemical parameters (concentration of NO_3-N, P, K, Ca, Mg, Na, Zn, Fe, Mn and Cu). Statistically significant correlations were found among several SQI. The data was subject to exploratory factor analysis using varimax rotation for principal component extractions. The Kaiser-Meyer-Olkin measure of sampling adequacy was 0.6, and the average communality was 0.71. Results showed that a 71% of the variance in the data was extracted into three components (soil erodibility, fertility, and reaction/salinity/biological) that showed distinct structures defined by the SQI measurements. Thus soil quality of the wastewater-irrigated field can be captured by smaller sets of measurements. However, further studies are needed that include more sites and a larger number of data points.

INTRODUCTION

The use of wastewater for agricultural purposes has recently become prominent in the arid and semi-arid southwestern U.S., especially due to the recurrent droughts and limited irrigation water availability from canals and deep wells. While the use of wastewater creates an opportunity for continued agricultural productivity in this region, there are concerns about how wastewater generated through different processes will affect soil quality, both in the short and long terms. However, specific metrics and protocols for soil quality assessment in lands receiving wastewater irrigation for crop production are lacking.

Soil quality is the capacity of soil to function and provide important ecosystem services to land users (Doran and Parkin, 1994). Soil quality cannot be measured directly, but depends on a selection of specific soil measurements for its characterization (Arshad and Coen, 1992). Soil measurements that can adequately characterize the health or quality of the soil are called soil quality indicators (SQI). Good indicators must be sensitive to soil management changes and, in many cases, more than one indicator may be needed to successfully assess and quantify the directional changes in soil quality. A combination of indicators that adequately characterize the changes in soil conditions constitutes the minimum data set (MDS) for a particular management goal (Andrews et al., 2002). A good MDS should integrate the physical, chemical and biological attributes of the soil (Idowu et al., 2008).

The land used for this study has experienced repeated application of reverse osmosis (RO) reject wastewater, to irrigate a mixed pasture over a period of two years. The objective of this study was to investigate the relationships among potential SQI measured in the wastewater-irrigated land using multivariate statistics, in order to detect if meaningful data structures exist within the measured indicators, and to reduce the number of measurements needed for soil quality assessment.

MATERIALS AND METHODS

The land that was studied was sown to a pasture mix (‘Hycrest’ crested wheatgrass [*Agropyron cristatum* × *desertorum*], intermediate wheatgrass [*Thinopyrum intermedium*], Lincoln smooth bromegrass [*Bromus inermis* Leyss.], Russian wildrye [*Psathyrostachys juncea]*, dryland alfalfa [*Medicago sativa* L.], and ‘Pauite’ orchard grass [*Dactylis glomerata* L.]), and irrigated with RO wastewater for two years. The electrical conductivity (EC) of the RO water was 2.73 dS/m, and the irrigation rate was 12.7 mm/week. This is in addition to the natural precipitation that was received (508 mm) over two years. The study site is located in northern New Mexico, and the soil is in the Blancot series, classified as Mesic Ustic Haplargids (USDA 1998). Forty-two surface soil samples (0-0.15 m) were collected, air-dried, and used for various analyses. Measurements performed included dry aggregate size distribution (Larney, 2008), with three parameters computed from these data: aggregates > 2mm (fraction of soil remaining on 2 mm and 4 mm sieves) [AGG > 2 mm], aggregates < 0.25 mm (fraction of soil collected in the bottom pan) [AGG < 0.25 mm], and the mean Weight Diameter (MWD), calculated using equation 1.

$$MWD = \sum_{i=1}^{n} x_i \left(\frac{w_i}{W}\right) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots equation\ 1$$

where x_i = average diameter of a size class; *i* = weight of aggregates in a size class of average diameter x_i;
W = total weight of sample; *MWD* unit is in millimeters.

Wet aggregate stability (WAS) was measured using the Cornell Sprinkle Infiltrometer, to apply water drops (2.5 J of energy) to air-dried soil aggregates (2-4 mm) for 300 s (Ogden et al., 1997). Soil texture was determined using the hydrometer method (Gee and Bauder, 1986). Permanganate-oxidizable carbon (POXC) was assessed using the technique developed by Weil et al. (2003). Soil organic carbon (SOC) was measured by the Walkley-Black method (Nelson and Sommers, 1982). Soil electrical conductivity and pH were measured on saturated paste extracts (Rhoades, 1996). Soil phosphate-P and the micronutrients Zn, Fe, Mn and Cu were extracted with ammonium bicarbonate-DTPA (Soltanpour, 1991) and plant-available Ca, Mg, and Na were extracted with ammonium acetate at pH 7 (Gavlak et al., 2003). All extracted metals were analyzed by Inductively Coupled Plasma-Optical Emission Spectroscopy. Nitrate-N (NO_3-N) was extracted with 1 M KCl (Mulvaney, 1996) and quantified colorimetrically using a cadmium-copper reduction column (Lachet Instruments, Milwaukee, WI).

Correlation analysis was performed to investigate linear relationships between different SQI measurements, after which an exploratory factor analysis (EFA) was performed on the data and rotated using the varimax method for principal components extraction. The purpose was to examine the pattern of SQI extraction into different factors, and to assess if the SQI that load high onto each factor component have meaningful structures that are related to soil processes. In addition, the Kaiser-Meyer-Olkin measure of sampling adequacy, Bartlett's test for sphericity (to establish whether correlations between variables significantly differ from zero or not), and communalities (the amount of variance accounted for by the variables) of the extraction were computed for the datasets.

RESULTS AND DISCUSSION

Correlation analysis showed that many of the potential SQI were inter-correlated, and Bartlett's test for sphericity was significantly different ($p<0.0001$) from the identity matrix, indicating that there is a scope for variable reduction in the datasets. However, not all of the soil measurements were included in the analysis. Out of the twenty-one measurements performed on the samples, nine of them were direct nutrient measurements, and including all of them in the analysis prevented the identification of a clear structure in the EFA. Therefore, all the secondary elements (Ca, Mg and Na) and the micronutrients (Zn, Fe, Mn and Cu) were excluded from the analysis. NO_3-N was also excluded because it is a highly variable indicator that changes rapidly both within and between seasons. One of the criteria for a good SQI is sensitivity to variations in management and stability between seasons (Idowu et al., 2008). Silt was the only soil texture indicator used because sand and clay were perfectly correlated.

Eleven indicators were finally used for the EFA: MWD, AGG > 2 mm, aggregates < 0.25 mm, WAS, SOC, POXC, EC, pH, % Silt, P and K. The Kaiser-Meyer-Olkin measure of sampling adequacy was 0.6, which is an acceptable value for conducting factor analysis with the datasets (Kaiser, 1974). The average of the communalities for the selected indicators was 0.71. Table 1 shows the eigenvalues and percent variance associated with each component extracted. Three significant factors were extracted based on the criteria of eigenvalue >1. The first component describes 33.6% of the variance associated with the data, while the second and third components accounted for 20.7% and 16.6% of the variance, respectively. The three components together accounted for a total of 71% of variance in the datasets.

Table 2 shows the breakdown of the SQI into different components. The first component is comprised of MWD, AGG > 2 mm, aggregates < 0.25 mm, and % silt, with MWD having the highest loading of 0.95. AGG > 2 mm and aggregates < 0.25 mm also had very high loadings, indicating that any of these SQIs could serve equally well for soil quality assessment and as a representative of this component. This first component can be described as the soil erodibility component, since most of the measurements are related to susceptibly of the soil to wind erosion. Silt content was also extracted into the first component, although with a lower loading. This was expected, since the silt-sized particles are also easily subject to wind erosion (Lyles and Tatarko, 1986).

The second component comprises K, SOC, and P, all with high loadings of >0.8 (Table 2). This component can be defined as the soil fertility component, since all these indicators are related to the supply of nutrients in the soil.

The third component consists of four indicators, and appears to be more diverse in functions than the two previous components, and pH had the highest loading in this component (Table 2). This component can be defined as soil reaction/salinity/biological component. The POXC is an indicator of labile carbon in the soil, and it is closely related to WAS and many soil biological measurements (Weil et al., 2003). Both POXC and WAS can be defined as soil biological indicators, while pH defines soil reaction, and EC defines the soil salinity.

Soil quality of the studied site can be assessed by selecting fewer measurements that are meaningful to soil processes, thus reducing cost of soil analyses (Idowu et al. 2008; Moebius et al. 2007). Results of the components extracted in this study show that representative soil indicators can be chosen from each component, thus eliminating the necessity to perform all the measurements in order to assess soil quality.

Based on Table 1, only one measurement each from components 1 and 2 would be required to capture soil function related to these components. For example, from component 1, either AGG > 2 mm or aggregates < 0.25 mm could be used, since they are easier to analyze and values are easier to compute compared to the MWD. From component 2, any of the three indicators can be selected, but SOC would be preferred because it integrates more than nutrients into its function. Component 3 is more complex, since many functions can be defined within this component. It would be preferable to select three SQI, based on the soil functions associated with this component. Selecting EC, pH and POXC from component 3 would be appropriate to capture the processes related to soil quality assessment. Thus, a total of five SQI out of the original eleven that were entered into the EFA would be sufficient to describe the state of soil quality of the studied land.

Although the use of EFA has been successful in identifying distinct SQI structures in relation to meaningful soil processes within this relatively small study, more samples are needed from soil in the region similarly irrigated with wastewater to verify our choice of SQI. Such a large-scale study may help to identify and validate the MDS necessary for rapid evaluation of the quality of soils that receive wastewater for agricultural production.

CONCLUSIONS

This study focused on detecting structures in datasets and reducing the number of measurements needed for assessing soil quality in a land that had received wastewater irrigation and sown to pasture in northern New Mexico. Exploratory factor analysis showed that there were distinct structures within the datasets, and these structures were meaningful and relevant to soil functional processes. Three components were extracted through the EFA. The first component could be described as the soil erodibility component, while the second could be described as the soil fertility component. The third component was mixed with respect to soil functions, and was described as soil reaction/salinity/biological component. Selecting five SQI was sufficient to describe the quality of the soil of the studied site. Expansion of soil sampling to include multiple fields with wastewater irrigation and different cropping systems is still needed, to develop and validate robust minimum data sets for assessing soil quality of wastewater irrigated fields in northern New Mexico.

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Table 1. Eigenvalues and percentage variance extracted through principal component analysis.

Component	Eigenvalues	% of Variance	Cumulative %
1	**3.699**	**33.631**	**33.631**
2	**2.281**	**20.736**	**54.368**
3	**1.830**	**16.639**	**71.006**
4	0.921	8.368	79.374
5	0.620	5.639	85.014
6	0.520	4.729	89.743
7	0.460	4.180	93.923
8	0.327	2.971	96.894
9	0.195	1.771	98.665
10	0.146	1.325	99.989
11	0.001	0.011	100.000

Table 2. Rotated component matrix after component extraction†

Soil quality indicator (units)	Component 1	Component 2	Component 3
Mean weight diameter (mm)	**0.950**	0.172	0.010
Aggregates > 2 mm (%)	**0.945**	0.172	-0.012
Aggregates < 0.25 mm (%)	**-0.805**	-0.102	-0.310
Silt content (%)	**0.687**	-0.042	-0.085
Extractable potassium (mg/kg)	-0.090	**0.891**	0.004
Soil organic matter (%)	0.200	**0.817**	-0.038
Extractable phosphorus (mg/kg)	0.140	**0.808**	-0.241
pH	0.103	0.261	**-0.745**
Permanganate-oxidizable carbon (mg/kg)	0.223	0.473	**0.682**
Electrical Conductivity (dS/m)	0.265	-0.169	**0.661**
Wet aggregate stability (%)	-0.463	-0.002	**0.597**

†Extraction was performed using the varimax method.

Constructed Wetlands and Planted Sludge Drying Beds for Decentralized Integrated Wastewater Management.

Manoj Pandey, Norwegian University of Life Sciences

ABSTRACT

Wastewater management by large centralised systems has proven to be expensive and difficult to operate in poor countries like Nepal. Constructed Wetlands (CW«s) are a wastewater treatment alternative that is simple in operation and cheap to construct using local means. The robustness and low maintenance requirements make such systems suitable for decentralized, community managed wastewater treatment. The performance of operating CW«s in Nepal indicate that they are able to reduce the organic pollutant load as well as the nutrients to an acceptable level. Constructed wetlands and sludge drying reed beds (SDRB) can be integrated to treat both wastewater and sludge. In order to develop design criteria for the CW«s and SDRB«s in Nepal two pilot scale experiments were conducted. One tested the performance of horizontal flow (HF) and vertical flow (VF) CW«s for wastewater treatment and compared planted and unplanted systems. The other experiment tested the performance of planted and unplanted sludge drying reed beds. In both experiments the planted beds performed better than the unplanted beds. For wastewater treatment the VF beds, or planted sand-filters, performed better than the HF beds. To meet Nepalese discharge standards HF beds are sufficient, but to meet stricter requirements a combination of HF and VF beds are recommended.

Willow Based Evapotranspiration Systems for the on-Site Treatment of Domestic Wastewater in Areas of Low Permeability Subsoils.

Laurence Gill, University of Dublin

ABSTRACT

Ireland has over one third of its population using on-site wastewater treatment systems, mostly consisting of septic tanks discharging effluent into a subsoil percolation area. However, across wide areas of the country the subsoil is of too low permeability for such systems. The aim of this research was to evaluate the use of evapotranspiration systems using willow trees in closed basins such that if sized correctly, they should produce no discharge either to ground or to surface water. 11 full-scale willow bed systems have been constructed as pilot trials at houses around the country in areas of heavy clay subsoils to treat the domestic wastewater produced. The systems were designed with variations between key parameters (effluent type, willow species, plan area, aspect ratio and effluent distribution), in order to determine sensitivity to these in terms of overall performance. The effluent flow into the basins, water level, rainfall and evapotranspiration have all been monitored over four years to determine the water budget and crop factors at each site.

Evapotranspiration results varied greatly between sites, with some sites exhibiting excellent willow tree establishment and correspondingly high evapotranspiration rates, while other sites showed the opposite performance. In general however, there was overflow from almost all the systems at different periods throughout the monitoring period, including some periods during the summertime. One significant problem identified was the usable void ratio in the basins that had been refilled with the excavated low permeability subsoil was much lower than expected thereby leaving little room for effluent storage over the winter periods. In addition, the evapotranspiration rates were lower when compared to other countries where these systems have been used, which was attributed mainly to the predominantly high relative humidity of the Irish climate. Water availability was determined to strongly influence the evapotranspiration rate from a system, while the addition of effluent was shown to have had a positive effect on willow tree development and evapotranspiration rates. Pollutant uptake / removal was found to be very high on the systems with well-established willow trees.

Using crop factors based upon these trial results, guidelines on the design and construction of willow systems have been prepared. As the achievement of a completely zero discharge system would appear to be difficult to achieve in an Irish climate, the guidelines were designed on the premise of minimising the number of overflow days while keeping within the boundaries of reasonable practical and financial constraints.

Developing an Extension Program on Onsite Septic Systems in Oklahoma.

Sergio M. Abit Jr., Oklahoma State University

ABSTRACT

Onsite septic systems are widely used in a state like Oklahoma wherein a large fraction of the population resides in rural areas that are not serviced by a city or municipal sewer system. Oklahoma is unique from many states because its soils and climate varies considerable from the eastern to the western border which causes a wide variety of systems to be utilized in the state. The state recently adopted soil profile characterization as a requirement for issuing installation permits and has just enacted a rather highly debated wastewater nitrate reduction policy. All these points to the need for an extension program to cater to such a critical industry. Recognizing this need, the Oklahoma Cooperative Extension Service launched a land-grant university-based extension program solely devoted on septic systems aimed at catering to the educational and technical needs of the state regulating agencies, certified installers, environmental advocacy groups and the different Native-American nations, among others. This presentation will illustrate the experience of establishing such extension program. A few details about the challenges, the early successes and the promise of running an upstart extension program will be presented.

Teaching Undergraduates the Basics of Decentralized Wastewater Treatment

David L. Lindbo, North Carolina State University

ABSTRACT

Many of our undergraduate soil science students at NC State University are employed in the decentralized waste water field doing soil and site evaluations and design of single family home systems for both public and private sectors. For 8 years we have been teaching a course that strives to provides both the book and field knowledge for them to succeed in their careers. Course materials are based on the CIDWT curriculums that have been modified for conditions and students in North Carolina. Course evaluations indicate a significant increase in students understanding of the subject. Students leave the course with basic skills that will need to be honed by on the job training.

Decentralized wastewater systems are here to stay and are an integral part of the overall wastewater infrastructure in North Carolina and the nation. Many of the graduates of the Soil Science Department at NC State University have spent at least part of their careers, if not all of it, evaluating sites and designing decentralized systems. Unfortunately, until recently they have had to learn most of the principles on the job. On the job training is valuable but it is not the most effective way to start a career.

Approximately 7 years ago the Soil Science Department redesigned one of its undergraduate courses, SSC 361 Environmental Soil Management, to focus in part on decentralized wastewater site evaluation and design. Shortly after this redesign a new major – Soil and Land Development – was approved. This new major embraces the fact that our graduates need more information on land use planning and specifically non-agricultural soil and land use. The redesigned course, SSC 361, fit well into this major and has been gaining popularity for the past few years (Table 1).

The syllabus (Table 2) was designed to accommodate 2 aspects of Environmental Soil Management that previously had not been taught and are not taught in any other undergraduate courses at NCSU (Table 2). The first 2/3 of the course is devoted to decentralized wastewater, specifically single family homes systems and site evaluation. The last 1/3 of the course builds upon the soil evaluation taught previously but focuses on sediment and erosion control primarily from construction sites.

The text for the Wastewater portion of the course utilizes several of the Consortium of Institutes for Decentralized Wastewater Treatment (CIDWT) publications as well as the Soil Science Society of America's Step-by Step Field Book (Table 3). These texts provide highly practical information that can be utilized by the students as they leave the academic setting and work as consultant. Since they are written for a national perspective and most of our students end up staying in North Carolina each student is provided with a copy of the North Carolina Rules for Onsite Systems. In addition to utilizing CIDWT's manuals the associated PowerPoint presentations (http://www.onsiteconsortium.org/resources.html) are also utilized although adapted for specific use in North Carolina in terms of systems types, soils and site conditions. Since these materials have been peer-reviewed and extensively field tested they provide a solid background for the course instruction.

In additional to lecture, laboratory and field exercises, students are required to work as a team to plan a decentralized wastewater system(s) for a small subdivision. This final exercise counts for 15% of their final grade. In addition to testing their knowledge of principles and rules it also gives most the students the first chance to work as a team and to synthesize knowledge and skills taught in the course. The parameters (Table 4) for the exercise are intentionally vague to force the students to ask questions of their instructor (client) as they would likely have to do once they are in the private sector. Results of the exercise are highly variable and are graded based on the student's explanation of what they did and why they did it as well as the overall calculations and adherence to the rules (Table 5).

Assessment of the course begins on the first day with a simple quiz designed to gauge the students overall understanding of wastewater treatment in general and decentralized wastewater treatment specifically (Table 6). The students are told that this first quiz does not count towards their grade so guessing or saying "I do not know" is acceptable. The questions from this quiz are answered throughout the course and reappear on the exams. Most students have some understanding of wastewater treatment from the outset but do not know the extent of decentralized usage nor whether decentralized is better than centralized system usage. By the end of the class they do have a better understand of how extensive decentralized systems are used (although regrettably they may not recall the exact numbers). Likewise they can make a more informed argument over the pros and cons of decentralized vs centralized system approaches to wastewater treatment

CONCLUSIONS

This course, although far from perfect, is filling a gap in the education of soil science students as well as students in related fields. The need for more extensive instruction in decentralized systems exists especially as one considers the number of systems both in North Carolina and nationally. The course could be improved by bring in more exercises with advanced treatment systems, operation and maintenance, inspection of existing systems , trouble shooting and more comprehensive economic analysis. In all likelihood this would require the creation of at least 1 to 2 more courses.

Table 1: Numbers of students enrolled in SSC 361. Overall numbers are increasing. Also of note is that the numbers of students outside the Soil Science major are also increasing.

Year	Students	Non-soil majors
2007	9	2
2008	7	2
2009	9	2
2010	8	2
2011	6	2
2012	7	3
2013	21	10
2014	16	9

Table 2: Generic SSC 361 Syllabus. Note the use of field sites for laboratory exercises.

Week	Monday	Lab – Monday PM	Wednesday
PART 1			
1	History of Wastewater	History of Wastewater	Wastewater Characteristics
2	Wastewater Treatment	Soil Morphology	Soil and Site Morphology
3	Soil and Site Morphology	Soil and Site Morphology	Soil and Site Morphology
4	Infiltration and Hydraulic Conductivity	Infiltration and Hydraulic Conductivity - Field	Water movement from trenches
5	Soil and Site Morphology – LTAR	Soil and Site Morphology – LTAR	System Types and Depths Requirements
6	System Types and Depths Requirements	Soil and Site Morphology - Field	EXAM 1
7	Subsurface Systems – Tanks and Gravity Distribution	System Layout - Field	Subsurface Systems – Distribution Components
8	Subsurface Systems - Advanced Treatment Systems	Subsurface Systems – Field	Subsurface Systems - Advanced Treatment Systems
9	Design Options	System Design	Surface Applications
10	Surface Applications	Surface Application - Field	EXAM 2
PART 2			
11	Basics of Erosion	Erosion Control Installation – Field	Erosion Processes
12	Modeling Erosion	Erosion Control Testing- Field	Predicting Storm Flow
13	Basics of Sediment Control	Sediment Control Installation – Field	Design of Sediment Control Systems
14	Basics of Turbidity Control	Turbidity Control Screening – Lab	Turbidity Control Chemistry
15	Developing an Erosion, Sediment, and Turbidity Control Plan	Tour Construction Site (as available)	Developing an Erosion, Sediment, and Turbidity Control Plan

Table 3: Text and references used in SSC 361.

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Lindbo, D.L. and N. E. Deal (eds.) 2005. Model Decentralized Wastewater Practitioner Curriculum. National Decentralized Water Resources Capacity Development Project. Consortium of Institutes for Decentralized Wastewater Treatment. North Carolina State University, Raleigh, NC

Logsdon, S. D. et al. (eds.) 2008. Soil Science: Step by Step Field Analysis. Soil Science Society of America, Madison, WI.

Table 4: Cover sheet for subdivision exercise.

The purpose of this exercise is to develop a preliminary site plan for land developer. The developer is interest in purchasing the land and has had the soils mapped on it. The soil scientist that did the mapping was unfamiliar with the systems and rules used in NC. The developer has hired your company to take the existing information and develop a scenario that will work best for her. She has given you the following parameters to consider:

1. The minimum lot size is ½ acre.
2. The lot must be 100 feet wide at the front of the house to lot lines.
3. The developer wants to set up the subdivision as a turnkey operation.
4. Each house is proposed to have its own well and septic system.
5. A roadway crossing the stream has been approved.
6. Scale of map is 1" = 50'

Your job is to develop a plan for the subdivision.

1. Use the .1900 rules to layout the system on paper.
2. You will need to use the soil and site information to establish loading rates and determine usable areas for septic systems.
3. You will need to propose roads and other infrastructure as needed.
4. You will propose lot lines and house location (assume 3000 ft^2 for 4 bedroom, 2250 ft^2 for a 3 bedroom).
5. You will need to justify all choices from the stand point of:
    a. the rules,
    b. the environment, and
    c. economics
6. Estimated costs for systems
    a. Gravity $3000 to 5000
    b. Pump $5000 to 8000
    c. Advanced pretreatment system $15000
    d. Drip dispersal $7000
    e. Other costs upon request
7. Building lot sells for $100000.
8. Any changes to the parameters given must be explained.
9. Connecting to central sewer is not an option.

Report:

Describe your evaluation process, choices made etc.
Show lots, houses, well(s), easements, roads, driveways
Show usable soil areas
Show LTAR(s) and how they were calculated
Show where system(s) are to be located
Describe each system(s) parameters

Type of system
Drainfield size

Table 5: Grading rubric for subdivision exercise.

Team Members ____________ ____________ _____________ _____________

1. Introduction
    a. Purpose (5 pts.)
    b. Scope (5 pts.)
    c. General site description (5 pts.)

2. Methodology
    a. Source material (5 pts.)
    b. Field work (5 pts.)
    c. System Choices (5 pts.)
    d. Compilation of materials (5 pts.)

3. Results
    a. Soil and site evaluation (40 pts.)
        i. Profile descriptions and LTAR
        ii. Map of usable soil
    b. Lot configuration (10 pts.)
    c. System design (10 pts.)

4. Summary (5 pts.)

Table 6: Introductory Quiz

Introductory Quiz

What is wastewater?

Why do we treat wastewater?

What are the treatment processes involved?

What percent of NC uses onsite wastewater systems?

What percent of the US uses onsite wastewater systems?

Which is better onsite systems or centralized sewer systems? Explain

Septic System Improvement Estimator.

Sara Heger, University of Minnesota

ABSTRACT

The Septic System Improvement Estimator is a spreadsheet-based model developed in Minnesota for the Board of Water and Soil Resources (BWSR) which estimates annual pollutant loads from problematic septic systems and accounts for the benefits of a range of septic system improvement, educational efforts and programs to identify the problematic systems. This paper will discuss why it was developed, the background data that supports the removal estimates, how to use the tool and the resulting data.

Certification Programs for Inspection of Onsite Wastewater Systems at Time of Sale: The Missouri and Iowa Experiences

Randall J. Miles*, James Gaughan, and Daniel Olson

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ABSTRACT

Programs for the certification of professionals to perform assessments of existing onsite wastewater systems at time of sale have been developed in Missouri and Iowa to assist in getting failing systems repaired, protect homebuyers, and protect the environment and public health. The Missouri program consists of two assessments: 1) an inspection and 2) evaluation. The inspection is the more comprehensive of the two assessments in that digging, taking measurements, performing a hydraulic loading test and a possible tracer dye assessment, as well as exposure of system components are mandated. The evaluation involves a visual and sensory walk over the system. The Iowa program involves a comprehensive assessment similar to the Missouri inspection. Both inspection assessments possess forms focused on specific system components and technologies. An assessment of a system in Missouri is initiated by the buyer or lender, while in Iowa every system must be inspected prior to transfer of deed. Prerequisites, certification course content, testing, plus continuing education requirements for both programs are presented in this paper, as is the number of systems assessed in each state.

Inspection of onsite wastewater systems has gained great interest in many communities and states in recent years. In many rural states, such as Missouri and Iowa, the first-generation onsite wastewater systems that were installed are currently termed substandard for a variety of reasons. These long existing systems are common place in many areas because 1) the technology at the time of installation was very limited; 2) population density of many areas has been very small; 3) dilution was considered by some regulatory authorities as acceptable; and 4) systems were put in on a temporary basis because it was perceived that central or municipal sewer would be coming in the future.

As many existing rural settings have become more densely developed, the population has become more aware of many environmental water issues, and lending institutions desire assurance of an adequately functioning onsite wastewater system before lending money to a potential homeowner. These situations have placed greater emphasis on development of inspection programs for these systems at the time of sale. In recent years onsite wastewater systems have gained greater consideration as there is less funding for development of municipal trunk lines to service greater numbers outside the municipal areas. The Environmental Protection Agency (USEPA, 1997) has designated onsite wastewater systems as a permanent solution to part of the void of some areas having access to municipal sewers. Although limited technologies were initially available, there has been a large expansion of available technologies for various soil-site receiving environments, as well as increased research and better understanding of these technologies for appropriate application and sizing to marginal sites. With greater awareness of water quality and quantity issues in many communities, properly sized, designed`, and maintained onsite wastewater systems can be a sustainable solution for recycling and reuse of water and nutrients within local watersheds and landscapes.

Inspection codes for onsite wastewater systems at time of sale have been developed for a wide spectrum of reasons, including: to provide renovation of failing systems; protection of homebuyers, particularly those who have not been served by an onsite wastewater system previously; protection of the environment and public health; and as a systematically “soft” way of eliminating illegal septic systems with a minimum of intrusion to the current homeowner. Overall, a comprehensive assessment of existing onsite wastewater systems at time of sale leads to a higher quality of life in the community while maintaining or increasing property values.

The Missouri Onsite Wastewater Loan Inspection and Evaluation program was initiated in 1998 with mixed results, and was later revised after input from a state-wide stakeholders committee (Miles and Roberts, 1999; Miles et al., 2004). This revised program involves two different assessments: inspection and evaluation protocols. An inspection is the more thorough, comprehensive assessment in which measurements must be made, with access to the various onsite wastewater components such as sewage tanks, through unearthing buried or hard-to-get-to components of the systems (Miles et al., 2004). Measurements and dimensions of the tank and soil absorption field must be performed, along with a hydraulic loading (formerly called a stress) test using a tracer dye in some cases (Miles et al., 2004). In contrast, the evaluation is less intrusive in that the professional assessor only “walks over” the system to get a visual impression of the assumed system, while also noting any smells or other evidence of a possible system failure.

Miles et al. (2004) provided the topical outline for the two-day Missouri Onsite Wastewater Loan Inspection and Evaluation Course. The course content has stayed very similar since 1999, with the exception of information and slides on new technologies which have recently been allowed for installation in Missouri. A prerequisite for the course is successful completion of the two-day basic installer course, which is taught by the Missouri Department of Health and Senior Services. This course is fundamental to present the basics of onsite wastewater systems for installers as well as others, such as existing home inspectors who desire to inspect onsite wastewater systems but have little experience with residential wastewater systems. This prerequisite is critical in that home inspectors typically make up 30-60% of the students enrolled in the class, with the rest being installers. The initial backgrounds of the home inspectors are excellent for providing comprehensive inspections and addressing liability issues, since these professionals are constantly applying these fundamentals to all aspects of their profession.

Major changes to the program since the description by Miles et al. (2004) have centered on a different reporting form format and associated submission to the Missouri Department of Health and Senior Services (MDHSS), as well as critical inspection point guides to specific areas of technology and specific propriety technologies. The first generation of inspection forms was in hard-copy (paper) format and utilized many of the same technical assessment points as outlined for operations and management by Consortium of Institutes for Decentralized Wastewater Treatment (CIDWT, 2005). Current forms use the same inspection points and are in electronic spreadsheet or pdf form files, which provide the ability to electronically submit the file to the MDHSS, although an inspector can also submit the files by mail. The individual forms currently used are: OWTS Assessment Summary (Fig. 1); Aeration Treatment Unit (ATU); Bio-Media Treatment Unit; Dispersal Field; Holding Tank; Lagoon Inspection; Pump Tank; Septic/Trash Tank; Setback Form; Site Diagram; Water Supply; and Wetland. With electronic submission of forms, the overall summary form, now called the OWTS Assessment Summary, is formatted such that the submission is not accepted or complete if all other pertinent, associated accessory technology forms are not submitted in concert with the OWTS Assessment Summary form. For example, if the OWTS Assessment Summary form is marked that the assessed OWTS has an ATU and drip distribution soil absorption field with a pump tank, the appropriate forms for the ATU, drip dispersal field, and pump tank, along with the setback and water supply form, are not submitted with the OWTS Assessment Summary form, the submission is not accepted. Also, if an extra, unrelated accessory form is submitted with these forms, the submission is not accepted. The number of assessments in Missouri since 2008 is shown in Table 1.

Critical item guidelines have been developed for specific technological components for some of the forms. These guidelines have been produced for ATUs, bio-media treatment units, septic/sewage tanks, holding tanks, lagoons, vegetative submerged wetlands, and water supply sources. These guidelines have been developed in concert with manufacturers of proprietary products as well by the regulatory and training team to provide a more consistent assessment of critical component assessments.

The Iowa time of transfer inspection program was initiated in April 2008 when Senate File 261 (SF 261) was passed, and a requirement for inspections at time of sale took effect on July 1, 2009. The reasons for developing the Iowa time of transfer inspection program included: getting failing onsite wastewater systems repaired; protecting home buyers; protecting the environment and public health; systematically eliminate illegal onsite water systems; providing secondary treatment; preventing systems from being “grandfathered”. Much of the legal basis for SF 261 was the Iowa Groundwater Hazard Statement, a document required for all property transfers. This statement is used as a disclosure method of potential environmental problems to buyers such as UST’s, wells, landfills, burial sites, hazardous waste sites and now, onsite wastewater systems. This statement is part of the onsite wastewater system inspection enforcement mechanism, and includes binding agreements that are now Iowa Department of Natural Resources (IDNR) forms.

This inspection program requires that every building or home with an onsite wastewater system must have it inspected prior to the transfer of deed. The larger number of inspections since inception (Table 2) compared to Missouri may be a result of the mandatory nature of the Iowa program. SF 261 is structured such that the county recorder cannot record the deed of conveyance documents without proof of inspection or binding document for inspection. The inspection process includes: all seller-financed real estate contracts, sales of one to four homes (larger numbers constitutes a public system), businesses with onsite wastewater systems, and buildings on leased land. The exempted systems include: transfers pursuant to a court order via foreclosure or forfeiture, transfer by trustee in bankruptcy, transfer by eminent domain, transfer by a fiduciary in the execution of a trust, estate or guardianship, transfer between joint tenants, transfers made to a spouse or a person in the lineal line of consanguinity, transfer between spouses resulting from divorce, legal separation or property settlement, transfer of property that will be razed or demolished, transfers of five hundred dollars or less, transfers between family corporations, partnerships, LLPs; Limited Liability Corporations (LLCs) where the deed is given for not actual consideration other than for shares or for debt securities of the family corporation, partnership, LP, LLP or LLC. Other exempted transfers made effective July 1, 2010 include: transfers of a property with a septic system installed within the last two years, transfers arising from the partitioning process, and transfers from a tax sale deed by the county treasurer.

SR 261 requires time of transfer inspectors to be certified by IDNR, and the inspector must use a uniform inspection protocol in concert with a standard inspection form (Table 3). Individuals eligible to become certified time of transfer inspectors must have two years of experience in the operation, installation, inspection, design, or maintenance of private sewage disposal systems. Individuals lacking this experience must complete pre-requisite courses, such as "Onsite Basics 101" and "Alternative Systems" offered by the Onsite Wastewater Training Center of Iowa, before attending the two-day inspection course with testing. The Iowa inspection form encompasses critical points of all components of the onsite wastewater system similar to those outlined for maintenance by CIDWT (2005), whereas Missouri uses individual inspection forms for specific components (e.g. tanks, soil absorption field, etc.) of the onsite wastewater system. In the protocol, tanks are required to be pumped and assessed at the time of inspection unless the tank was pumped, inspected, and sized by a licensed septic tank cleaner within the last three years. The seller must maintain documentation of this activity. Lids to the tanks and distribution box must be uncovered for assessment. The hydraulic loading test and/or probing of the soil absorption field must also be performed. Additionally, indoor plumbing must be checked to assure that it is connected to the onsite wastewater system, and other important components of the system must also be uncovered for inspection.

The fundamental philosophy of the Iowa inspection criteria is that an onsite wastewater system with primary and secondary treatment that is not creating an environmental or public health hazard should "pass". Therefore, all systems must have primary and secondary treatment components. The secondary treatment component assessed could be any of the following: soil absorption trench, sand filter, media filter, aeration treatment unit, or wetland system. In situations where the onsite wastewater system would have a sewage tank and/or field/filter too small, or the soil treatment area is too close to a limiting feature such as groundwater or bedrock, the system does not have to meet the current code if it has a secondary treatment technology that is working properly. Completed inspections are valid for 2 years.

The duties of the inspector are to report what they saw on the day of the inspection, failed and illegal systems and the need for repair, an onsite wastewater system is functioning but needs components repaired, and a system that is functioning when inspected but may be undersized. The professional inspector does not function as an operations and maintenance professional, but solely as an inspector. The number of inspections in Iowa since program inception is provided in Table 2.

SUMMARY

Missouri and Iowa have developed onsite wastewater inspection programs to assess the status of onsite wastewater systems servicing existing homes that are changing ownership. The Missouri program involves two types of assessments: an inspection and an evaluation. The inspection is the more comprehensive assessment in that lids of all technical components are removed for assessment, and measurements such as pump drawdown must be performed. The evaluation is just a "walk over" of the system in which a visual and smell sensory assessment (without removing lids and taking measurements) is performed. With both Missouri assessments the program is voluntary in that it is normally initiated by the buyer, the buyer's agent, or the buyer's lender.

The Iowa time of transfer program inspection is not voluntary, as all systems (with some exceptions) are required to undergo inspection at the time of transfer. This program is comprehensive in that measurements and observations must be made with access to various wastewater components. While both state programs have many of the common goals of for both the functioning of the individual onsite wastewater systems and betterment of the community and environment, the Missouri program places the decision about any action to correct system malfunctions and deficiencies on deliberation between the buyer and seller. The Iowa program mandates updating the system in a timeframe set by the county Board of Health. The cost is negotiated between buyer and seller. The number of assessments of existing systems for each state is the reflective of the regulatory nature of each program. The Missouri program, which is voluntary, has fewer combined inspections and evaluations than the Iowa program, which is essentially required for all existing onsite wastewater systems.

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Table 1. Number of OWTS inspections, re-inspections, evaluations, and total number of assessments performed in Missouri, 2008-2013.

Year	Inspections	Re-inspections	Evaluations	Total
2008	821	5	268	1094
2009	770	19	420	1209
2010	939	26	272	1237
2011	960	30	271	1261
2012	1361	40	239	1640
2013	1254	47	212	1513

Table 2. Number of OWTS inspections in Iowa, 2009-2013.

Year	Inspections
2009	2500
2010	3850
2011	4200
2012	4550
2013	4900

Table 3. Information Reported on the Iowa Time of Transfer Inspection Report 542-0191.

Primary Treatment	**Information Needed**
Septic Tank	Size; material; condition; tank pumped?; date; licensed pumper
Trash/Processing Tank	Size; material; condition; tank pumped?; date; licensed pumper
Aerobic Treatment Unit	Manufacturer; size; tank pumped?; date; licensed pumper; maintenance contract?; expiration date; service provider
Pump Tanks/Vaults	Type; size; condition
Distribution System	Distribution box; outlets used; condition; header pipe(s); number of lines; pressure dosed?
Secondary Treatment	
Absorption Field	Length (determined by); conditions of field (determined by); type of trench material
Sand Filter	Size (determined by); vent pipes above grade; discharge pipe located; effluent sample taken; results of sample
Media Filters	Type; maintenance contract; expiration date; service provider; condition
NPDES General Permit No. 4	Required; permitted; NOI provided;
Other Components	Alarms-working?; Disinfection-working?; Control Box; Timers; Inspection Ports; list others not mentioned
Overall Condition of the Private Sewage Disposal System	Report System status; Explain; Comments

Figure 1. Copy of the Missouri OWTS Assessment Summary Form.

ONSITE WASTEWATER TREATMENT SYSTEM (OWTS) ASSESSMENT FOR REAL ESTATE TRANSACTIONS

Date of Assessment: ______ Assessment Type: Evaluation Inspection Re-Inspection DHSS FILE#:

Owner's Name: ______

Site Address: ______

City, ______ MO State Zip

Latitude: ______ Longitude: ______ Requesting party: ______

County: ______ Lot size: ______ acres Contact Telephone#: ______

Owners: It is not necessary to contract with the inspector to make recommended repairs.

FACILITY INFORMATION

Type of Facility:

Residence

Single

Multi-family-shared

No. of Bedrooms:

No. of Occupants:

Check All That Apply:

Garbage disposal

Jetted/oversized tub

Shower tunnel

Water Softner

Business

Type:

No. Of units:

SYSTEM HISTORY

Approximate Age of OWTS: ______ years

System was permitted:

NA Yes No

Repairs made to OWTS: Date

HYDRAULIC LOAD TEST

System has been in use for at least 6 months:

Yes No

If vacant, number of days vacant:

30 days or less

31 to 60 days

More than 60 days

If vacant more than 60 days, or if time vacant is unknown, system shall not be subject to hydraulic test.

Hydraulic test performed Yes No

Dye introduced Yes No

Introduction of dye is not mandatory

WATER SUPPLY

Private Supply Yes No

Wellhead construction criteria:

Met Not Met

Water sample date:

Water sample results:

Acceptable Unacceptable

Resample:

1st resample date:

Water resample 1 result:

Acceptable Unacceptable

2nd resample date:

Water resample 2 result:

Acceptable Unacceptable

TREATMENT/DISPERSAL

OWTS components:

ATU Wetlands

Septic tank/Trash trap

Lagoon Holding tank

Pump/processing tank

Bio-filter (select media):

Sand filter Peat Filter

Textile Foam

Other:

Dispersal system (select type):

Conventional

LPP Drip

Mound At Grade

Discharge Pipe (Unacceptable)

☑ Setback Form OWTS Evaluation

Detail inspection/evaluation forms are to be attached for boxes checked above.

LICENSED INSPECTOR/EVALUATOR SIGNATURE

Private Inspectors/Evaluators are Licensed by the Department of Health & Senior Services

The information contained herein is a complete and accurate assessment of the OWTS on the date of this assessment and does not guarantee the continued functioning of this system.

Print Name: ______ ID Number: ______

Signature: ______ Job No.: ______

ASSESSMENT SUMMARY

Set back distances are: Met Not Met

INSPECTIONS - As reported in the attached forms, inspection criteria are:

Met Not Met

EVALUATIONS - As reported in the attached forms, evaluation criteria are:

Acceptable Unacceptable Undeterminable

TYPE OF DEFICIENCY: Component Surfacing Effluent

DHSSREV12/00

Onsite and Decentralized Systems Water and Wastewater Engineering: Course Development and Delivery Experiences in Higher Education

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ABSTRACT

For nearly a generation now, the virtues and benefits of onsite and decentralized infrastructure for water and wastewater have been widely recognized and approaches, devices, and technologies have been advocated as critical components for a 21st Century water infrastructure in the United States. Yet, the principles and practices of modern onsite and decentralized infrastructure have not been broadly incorporated into curriculum within higher education. Undergraduate and graduate students have received education concerning onsite and decentralized systems through research experiences while pursuing a degree, through listening to guest lectures in their classes or a seminar series, by sitting in on shortcourses offered at their university, or attending technical conferences. However, the vast majority of students have not had access to semester-long courses focused on onsite and decentralized systems that serve as electives or required courses for juniors/seniors and graduate students. A few universities have been successful in developing semester-long courses that are focused on engineering and design of onsite and decentralized systems and which have been sustainably delivered. There appear to be several keys to achieving this: 1) having a faculty member who can champion development and delivery of a course, 2) having that same faculty member have time and support to develop and establish the course; 3) having a receptive administration that will recognize the value and need for a course and support sustained delivery of it, 4) having credit-hour space in one or more degree programs to make the course a valued elective, and 5) stimulating interest in the course and attracting an adequate number of students to enroll in it. This paper summarizes the status of educational efforts at multiple universities and provides details on a course developed and delivered over the past nine years at the Colorado School of Mines. Lessons learned are highlighted and recommendations are made for course development and sustained delivery.

INTRODUCTION

Water and wastewater infrastructure in the United States evolved during the 20th Century based on major investments at the Federal and state level. At the end of the 20th Century, approximately 80% of the population was served by centralized infrastructure with 20% served by onsite and decentralized systems. Near the end of the 20th Century and into the 21st, a series of activities and events in the United States helped catalyze a new paradigm involving onsite and decentralized approaches, technologies, and systems for wastewater treatment and water reclamation and reuse. There was growing interest in how onsite and decentralized systems could help provide more sustainable infrastructure by: 1) reducing the use of drinking water to flush toilets and transport waste to remote wastewater treatment plants, 2) preventing pollutant discharges from large centralized systems including sanitary sewer overflows, combined sewer overflows; and leaking sewers, 3) recharging water near the point of water extraction and avoiding water export and depletion of local water resources, 4) enabling recovery and reuse of water, organic matter, and nutrients (N, P, K); 5) lowering consumption of energy and chemicals, and reducing greenhouse gas emissions, and 6) providing infrastructure that is more robust and resilient to natural disasters and climate change.

Based on major research and development efforts over the past two decades or more, modern onsite and decentralized systems have evolved to include a growing array of approaches, devices and technologies. Ultra efficient fixtures and source separation plumbing can enhance water infrastructure by minimizing water and energy demands and enabling reuse. Treatment can be

Source: Siegrist RL. 2014. Onsite and decentralized systems water and wastewater engineering: course development and delivery experiences in higher education. In: Innovations in Soil-based Onsite Wastewater Treatment, Proc. Soil Society Society of America Conference, Albuquerque, NM, April 6-7, 2014. 13 pp. (29Apr14)

achieved using anaerobic and aerobic bioreactors, porous media biofilters, sorbent filters, membrane separation units, constructed wetlands, soil treatment and landscape dispersal units, and other technologies. Reuse of reclaimed water can occur through garden and landscape irrigation, toilet flushing, and other applications. Sensors and monitoring devices can be used to verify and enhance performance and enable remote process control and system management to monitor and automatically correct any system malfunction. Systems can mimic natural processes to achieve performance objectives while minimizing water, energy and chemical use, and enabling beneficial reuse. Onsite systems can be applied at the building-scale while decentralized systems can be used at the development-scale.

Applications of onsite and decentralized systems span rural, peri-urban, and urban areas in industrialized nations like the United States. However, beyond applications in industrialized nations, onsite and decentralized systems are critical to providing safe drinking water and adequate sanitation in developing countries. In developing countries worldwide, concerns about sustainability of large water and wastewater infrastructure are not yet paramount. Rather, concerns are focused on how best to provide to simple solutions for safe drinking water and effective sanitation - solutions that are effective, affordable and socially acceptable. Providing safe drinking water and adequate sanitation are critical to achieving the Millennium Development Goals established by the United Nations.

For nearly a generation now, the virtues and varied benefits of onsite and decentralized systems have been widely recognized and approaches, devices, and technologies have been advocated as critical components for a 21st Century water infrastructure in the United States and abroad (e.g., WERF, 1999; Siegrist, 2001a,b; Tchobanoglous, 2002; Nelson, 2003a,b; Jenssen et al., 2004; Daigger, 2008; WERF, 2009; Sydney, 2012). Translating this recognition and advocacy into meaningful impacts requires a portfolio of education and training activities that target different audiences to achieve different outcomes. This paper highlights the context for education and training focused on onsite and decentralized systems and explores the development and delivery of semester-long courses in support of degree programs in higher education.

HISTORICAL CONTEXT AND ENABLING DEVELOPMENTS

Education and training concerned with onsite and decentralized systems has been available for decades and it has been expanding during the past few years. Varied activities have served different audiences to achieve different outcomes. The highest level of effort and activity has occurred in the education and training of practitioners involved in soil and site evaluation, onsite system installation, and operation and maintenance. For example, the National Association of Wastewater Technicians (NAWT) has education and training programs that are offered periodically including installer training, operation and maintenance training, and inspector training and certification (NAWT, 2014). The National Onsite Wastewater Recycling Association (NOWRA, 2014) offers an annual conference and education and training in these areas. State organizations, often affiliated with NOWRA, offer education and training conferences (e.g., CPOW in Colorado (CPOW, 2014)). Universities, particularly land grant institutions, offer a range of continuing education programs for practitioners and regulators (e.g., Univ. of Rhode Island, North Carolina State Univ.). All of these efforts are extremely valuable as they help improve the state of practice and increase the professionalism of the field.

Beyond education and training of current practitioners and regulators, education of undergraduate and graduate students who will become design professionals and policy makers of the future is also critical to help foster necessary and appropriate changes in water and wastewater infrastructure. During the latter half of the 20th Century and up until today, education of undergraduate and graduate students concerning the science and technology of onsite and decentralized systems has occurred as a result of their involvement in research activities during completion of an M.S. or Ph.D. degree. This was the case for the author during completion of his M.S. and Ph.D. degrees in Civil and Environmental Engineering within the Small Scale Waste Management Project (SSWMP) at the University of Wisconsin during the 1970s and 80s. During that period there was a large, robust program of research within SSWMP and scores of students were involved working with numerous faculty and staff in Civil and Environmental Engineering, Agricultural Engineering, Soil Science, and other departments. There were numerous seminars to attend, guest lectures to be heard, continuing education shortcourses to sit in on, and modules within mainline courses periodically available. But there were no semester-long courses routinely offered that were focused on engineering of onsite and decentralized systems. This appeared to be the situation at most universities, including those with active research programs such as SSWMP. At relatively few universities were courses or major portions thereof being delivered. Most notable perhaps is a course that was offered at the University of Washington by the late Professor Robert Seabloom.

While research and educational activities related to onsite and decentralized systems had been ongoing for many years in the United States, the critical need for expanded efforts became clear near the end of the 20th Century. In 1997 Congress required the U.S. Environmental Protection Agency (USEPA) to prepare a report to Congress examining the appropriate use of onsite and decentralized systems (USEPA, 1997). This report concluded that "adequately managed decentralized wastewater systems are a cost-effective and long-term option for meeting public health and water quality goals" and identified five primary barriers to sustained widespread use, including 1) misinformation and limited knowledge about onsite systems, 2) legislative and regulatory constraints, 3) lack of system management, 4) existing engineering practices, and 5) restricted access to funding. To help overcome the barriers, Congress authorized funding of what became the National Decentralized Water Resources Capacity Development Project (NDWRCDP). The NDWRCDP sponsored workshops, applied research projects, and educational initiatives with an initial focus on decentralized wastewater systems. To help foster the needed education and training, faculty from nearly 20 universities formed the Consortium of Institutes for Decentralized Wastewater Treatment (CIDWT). During this same period, the Water Environment Research Foundation convened a landmark workshop involving academic, industry, and government leaders who were charged to re-evaluate the premises of wastewater treatment infrastructure and prepare a report: "Research Needs to Optimize Wastewater Resource Utilization" (WERF, 1999).

During the early years of the 21st Century, the NDWRCDP broadened its focus to encompass integrated decentralized water resource infrastructure**.** A national workshop in 2002 focused on distributed and nonstructural water and wastewater systems (Nelson, 2003a,b). Then in November 2005 and January 2006, the NDWRCDP convened a diverse group of environmental, engineering, utility, industry, and public interest representatives for a four-part workshop series to discuss new strategies for the advancement of integrated and decentralized water resource infrastructure in the U.S. It was increasingly clear that the policies and practices associated with traditional water supply and wastewater management in the United States were evolving in the

21st Century with growing concerns about resource efficiency, sustainability and resiliency. Onsite and decentralized systems involving water supply, wastewater management, and stormwater control were clearly necessary and appropriate within the 21st Century water infrastructure in the United States and abroad.

COURSE DEVELOPMENT AND DELIVERY

A variety of educational and outreach activities have occurred within higher education that have been focused on or related to onsite and decentralized water and wastewater systems. As noted earlier, education of undergraduate and graduate students has occurred through research experiences, seminars, guest lectures, and shortcourses. Continuing education efforts involving short courses and training sessions have been directed at contractors, design professionals and regulators. In some cases these efforts have been designed and implemented to support educational requirements for licensure to practice. Education of the general public has been accomplished through preparation and dissemination of informational brochures, pamphlets, and video clips. Above and beyond these efforts, this paper explores the nature and extent of semester-long courses that are being delivered at universities within the United States that are focused on onsite and decentralized systems. Of particular interest are courses that may be offered as requirements or electives in support of degree programs (B.S., MS., Ph.D.) in departments such as Civil and Environmental Engineering or Agricultural and Biosystems Engineering. In this section, a summary is presented describing the nature and extent of university-level course development and delivery.

University Curriculum Development by CIDWT. Recognizing the need for enhanced education and training, the NDWRCDP provided support to the CIDWT during the early 2000s to carry out a project to develop educational materials, including modules to support university curriculum (Gross et al., 2005). As stated on the CIDWT website (CIDWT, 2014):

> "Under the University Curriculum project, appropriate modules were developed for teaching a one-semester laboratory and field course in onsite/decentralized wastewater treatment and natural water reclamation systems. The target audience for the materials is third- and fourth-year engineering students. The modules can also be adapted for undergraduate and graduate-level university courses in Environmental Health and other non-engineering curricula. The University Curriculum is available on CD-ROM with a navigational and organizational macro. The format is such that the materials are accessible and modifiable using software that instructors will have readily available".

The materials developed under this CIDWT project (Table 1) were made available and did facilitate delivery of one or more modules within current course offerings or provide a starting point for development of a stand-alone course.

Course Development at Universities in the United States. Prior to, during and following the curriculum development efforts by CIDWT, stand-alone courses and portions thereof, were developed and delivered at several universities in the United States. To gain insight into course offerings and the history behind their development and delivery, an informal email query was made to faculty colleagues at universities across the United States. The email query stated:

> "I am doing a little informal checking to update myself with respect to the number of other universities that might be delivering a semester-long course focused on onsite and decentralized systems design and engineering. I am particularly interested in the extent to which this is being done for juniors and seniors and graduate students as an elective for a

> B.S. or grad degree in civil and environmental engineering or through another department. Does your university offer such a course? Do you know of any other universities that offer a semester-long course as noted above?"

This query was sent to faculty at 27 universities that were selected based on their history of involvement in research and education related to onsite and decentralized systems or their involvement in programs for water and sanitation for health (WASH). The email query was supplemented by searching the online course information systems at the universities. While this query and web-based inquiry were informal and by no means comprehensive, insights were gained concerning the status of course offerings with a focus on engineering of onsite and decentralized systems.

It appears that many universities have some aspect of their undergraduate and graduate curriculum that addresses onsite and decentralized systems (Tables 2 and 3). However, in most cases, this involves mentoring of students involved in research or delivery of one or several lectures on topical areas of interest to a faculty member within other mainline undergraduate or graduate courses (e.g., within courses on water supply and wastewater treatment). In a few cases, lectures on onsite water and wastewater systems have been delivered in courses within specialty programs (e.g., a WASH program).

At a few universities during the past decade, serious efforts have been made to develop a new course focused on onsite and decentralized systems. In some cases course development was started but for various reasons, the course did not get fully developed and established and never became a routine offering (e.g., Michigan State Univ.). In some cases a course was developed and approved for routine delivery, but after it was delivered once or a few times, it was no longer offered (Univ. of Arkansas, Univ. Tennessee). This was often the result of the faculty member responsible for the course having to deal with other demands on his/her time and effort or because the faculty member left the university. At a number of universities, faculty have had an interest in developing a course but simply have not had the time or support to do so (Table 4).

A few universities have developed and sustained a stand-alone course focused on onsite and decentralized systems (Table 3). These courses are generally offered within Agricultural and Biosystems Engineering (ABE) or Civil and Environmental Engineering (CEE) although they are often cross-listed in other departments (e.g., Soil Science, Natural Resources Science, Water Resources Management). Some courses are delivered through classroom lectures complemented by laboratory or field sessions (e.g., CSM, Univ. of Wisconsin - Madison, Univ. of Washington), while others are delivered as online courses (e.g., Univ. of Arizona). At the Univ. of Washington, a new course was developed to replace an older onsite wastewater course and the focus was changed to include decentralized systems, reuse and developing countries and attention was given to pedagogical methods for its delivery (Gaulke et al., 2008). Courses that are housed in engineering departments or programs generally satisfy ABET criteria (ABET = Accreditation Board for Engineering and Technology) and thereby support ABET-accredited degree programs.

<u>Course Development and Evolution at CSM.</u> A course focused on engineering of onsite and decentralized systems was developed at CSM in the early 2000s and has been offered every year for the past nine years (Table 4 and 5). The course titled, "Onsite Water Reclamation and Reuse", evolved out of perspectives gained by the author during the past 40 years. These perspectives are based on the author's experiences beginning with those during his graduate studies at the University of Wisconsin during the 1970s and 80s where he worked on research within SSWMP. During the next two decades, experiences were gained and observations were

made in the U.S. and abroad. After arriving at CSM in 1995, the Small Flows Program was initiated and research and educational activities over more than a decade have focused on advancing the science and technology of onsite and decentralized systems (Siegrist et al., 2013). During this period, the need for, and opportunity to achieve, development of a course that would help educate the next generation of design professionals, regulators, and policy makers became obvious. This was consistent with and connected to the recognition and efforts within NDWRCDP and CIDWT.

Course development efforts at CSM occurred by the author over several years and were completed in an initial form during a sabbatical in spring 2005. The syllabus for the course was developed, lecture materials in the form of slides and notes were prepared, and homework assignments, exam problems, and class project ideas were laid out. The course was offered for the first time during the spring semester of 2006 as a "special topics" course within the Division of Environmental Science and Engineering (ESE). The course number and name were "ESGN498A - Onsite Wastewater Reclamation and Reuse" and it was offered as a 3-credit course available to juniors and seniors as well as graduate students; nine students enrolled. The class was scheduled with 75-min classroom periods on Tuesday and Thursday afternoons during a 17-wk spring semester. The course was offered again in spring 2007 and spring 2008 under the same circumstances and 7 and 15 students enrolled, respectively. The course was sufficiently well received that in early 2008, it was proposed to CSM for formal approval for continued delivery as a university course. The course - "ESGN460 – Onsite Water Reclamation and Reuse" - was approved that semester. During the spring semesters of 2009 to 2013 the course has been offered as a 3-credit hr. semester-long course and enrollments have ranged from 15 to 30 with roughly 1/3 of those being graduate students and 2/3 being juniors and seniors. Due to a university reorganization where ESE was merged with Civil Engineering to form a new Department of Civil and Environmental Engineering (CEE), ESGN460 was renumbered as CEEN472 for spring 2014, but the course delivery has continued otherwise unchanged.

Beginning in 2006 and continuing until today, the course purpose and scope evolved along with the breadth and depth of material covered. Considerable efforts were directed at continuous improvements in technical content concerning principles and processes, development and inclusion of design equations, consistency in terminology and parameter definitions across topical areas, development of real-world problem and project assignments, and so forth. A primary goal was to evolve the course to a level in terms of technical underpinning and rigor so it would be on par with other course offerings available for juniors/seniors and graduate students pursuing degrees in CEE, Hydrologic Science and Engineering (HSE), Humanitarian Engineering (HE), and similar programs. Throughout the past nine years, while the breadth and depth of coverage has evolved, the focus of the course has remained fixed on the selection, design, and implementation of onsite and decentralized infrastructure for rural and urban settings that can achieve effective wastewater treatment as well as resource efficiency and reuse goals. Topics covered include: water use and wastewater generation, water use efficiency and source separation, alternative collection systems, engineered and natural treatment units, effluent dispersal and reuse options, resource efficient systems, and system performance assurance and management (Table 5). A set of more than 800 slides has been prepared along with a set of notes and these have been continuously updated based on course delivery experiences. These course materials along with other reference materials are distributed to all students enrolled in the class at the beginning of the semester. In years past this was done using a CD, but during spring 2014, the materials were distributed to students using a USB drive. The USB drive enables updates to occur during the semester and also supports dissemination of very large files. Learning has continued to be assessed through five design problems, two examinations, and a class project (which includes a written report and oral presentation). Students complete the design problems and class project in teams of 2 to 4 students each. The course is a technical or free elective within several degree programs including: Environmental Engineering Science, CEE, HSE, and HE. Consistent with ABET accreditation, CEEN472 course outcomes have been mapped against ABET (a) through (k) outcomes.

OBSERVATIONS AND LESSONS LEARNED

It appears that the principles and practices of modern onsite and decentralized systems have not yet been broadly incorporated into semester-long course offerings within higher education. Most courses related to water supply and wastewater management that are included as requirements or electives within undergraduate and graduate engineering programs at universities throughout the United States continue to focus on engineering of larger centralized infrastructure. In some respects, this is unfortunate in that it reinforces the design and implementation of the same large centralized infrastructure of the 20^{th} Century, which is being challenged today as unsustainable.

In response to the query of faculty colleagues at other universities, several comments were shared re: their experiences and lessons learned. A summary of these is given in Table 4. Observations and lessons learned by the author during delivery of “Onsite Water Reclamation and Reuse” at CSM are summarized in Table 6. Based on the responses received from other faculty as well as the author’s own observations, a general theme concerning the keys to the sustained successful delivery of a semester-long course in onsite and decentralized systems appear to include: 1) having a faculty member who can champion development and delivery of a course, 2) having that same faculty member have time and support to develop and establish the course; 3) having a receptive administration that will recognize the value and need for a course and support sustained delivery of it, 4) having credit-hour space in one or more degree programs to make the course a valued elective, and 5) stimulating interest in the course and attracting an adequate number of students to enroll in it.

SUMMARY AND CONCLUSIONS

Undergraduate and graduate students have received education concerning onsite and decentralized systems through research experiences while pursuing a degree, through listening to guest lectures in their classes or as part of a seminar series, by sitting in on shortcourses offered at their university, or attending technical conferences. However, the vast majority of students have not had access to semester-long courses focused on onsite and decentralized systems that serve as electives or required courses for juniors/seniors and graduate students. Courses that were developed and offered in the past for a period of time (e.g., one or a few deliveries) appear to have been the result of an inspired faculty member and his/her efforts, and in his/her absence (responding to other commitments or retiring or otherwise leaving the university), the course tends to wither and eventually get dropped. Inspired faculty who would like to develop a semester-long course have not been able to for various reasons. Notably, faculty efforts at many universities, particularly land grant schools, have been directed at continuing education and outreach for contractors, designers, and regulatory officials, often to fulfill educational needs for licensure to practice. Some view this as equally important to delivering courses for students in higher education since they think it will have a near-term impact on the state of practice.

Several universities have been successful in developing semester-long courses that are focused on engineering and design of onsite and decentralized systems and which have been sustainably delivered. As noted earlier, there appear to be several keys to achieving this: 1) having a faculty member who can champion development and delivery of a course, 2) having that same faculty member have time and support to develop and establish the course; 3) having a receptive administration that will recognize the value and need for a course and support sustained delivery of it, 4) having credit-hour space in one or more degree programs to make the course a valued elective, and 5) stimulating interest in the course and attracting an adequate number of students to enroll in it.

To foster improved dissemination of knowledge to students in higher education about modern onsite and decentralized systems and approaches, devices and technologies several steps can be taken. Guest lectures can readily be included in existing courses or in seminar series. Modules concerning relevant topics of special interest to one or more faculty can be integrated into other courses focused on water and wastewater, energy, water resources, sustainable design, etc. Onsite and decentralized systems analysis and design can be included as projects in senior design courses and field sessions. All of these efforts are being done at several universities, including CSM, and this appears to be a good step toward building awareness and understanding and laying the groundwork for a new semester-long course or supporting one that already exists. However, developing and establishing a semester-long course focused on onsite and decentralized systems that will be successful and sustained in delivery is a necessary and appropriate goal. Education of undergraduate and graduate students who will become design professionals and policy makers of the future is critical to help foster necessary and appropriate changes in water and wastewater infrastructure. Course development does take a major effort over a period of time under the right circumstances. Setting up a course to serve degree programs in multiple departments on campus – e.g., Civil and Environmental Engineering, Agricultural and Biosystems Engineering, Hydrologic Science and Engineering, etc. - is a sound strategy to help garner support and build enrollments. It is also recommended that faculty take advantage of resources available to aid development and delivery of a new course (e.g., CIDWT curriculum, course materials available from the author and others, educational literature).

ACKNOWLEDGMENTS

I wish to acknowledge numerous faculty who provided thoughtful responses to my query for information including: Professors Kitt-Farrell Poe (Univ. of Arizona), Findlay Edwards (Univ. of Arkansas), Joe Yelderman (Baylor Univ.), Kara Nelson (Univ. of California – Berkeley), George Tchobanoglous (Univ. of California – Davis), Scott Summers (University of Colorado), David Radcliffe (Univ. of Georgia), Ted Loudon, (Michigan State Univ.), Larry Stephens (Stephens Consulting Services, affiliated with Michigan State Univ.), Sara Heger (Univ. of Minnesota), Randy Miles (Univ. of Missouri), Robert Rubin (North Carolina State Univ.), Nancy Deal (NC Dept. of Health and Human Services, formerly with North Carolina State Univ.), Dave Sabbatini (Univ. of Oklahoma), Sergio Abit (Oklahoma State Univ.), Art Gold and George Loomis (Univ. of Rhode Island), John Buchanan (Univ. of Tennessee – Knoxville), Dave Stensel (Univ. of Washington), Linda Strande (Eawag in Switzerland, formerly with Univ. of Washington), and Jim Converse and K. Karthikeyan (Univ. of Wisconsin – Madison). I am also grateful to Kathryn Lowe (CSM) who reviewed the draft manuscript and provided valuable comments.

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Table 1. Summary of the CIDWT curriculum development materials (Gross et al., 2005; CIDWT, 2014).

Curriculum area
I. Fundamental concepts for environmental processes
II. Site evaluation
III. Wastewater characteristics
IV. Treatment processes: A. Onsite nitrogen removal; B. Septic tanks; C. Media filters for wastewater treatment; D. A critical review of wetland treatment processes; E. Constructed wetlands; Design approaches; F. Aerobic treatment units; G. Disinfection; H. Soil treatment systems
V. Distribution and dispersal systems: A. Effluent conveyance; B. Drip dispersal; C. Spray dispersal; D. Water reuse systems
VI. Hydraulics and controls: A. Hydraulics; B. Instrumentation and controls
VII. Septage

Table 2. Curriculum focused on onsite and decentralized systems at universities in the U.S.[1]

University		**Semester-long course is offered that is focused on the subject[2]**	
University	Affiliation of responding faculty	No	Yes[3]
Baylor University	Department of Geology	N	
Colorado School of Mines	Civil and Environmental Eng.		Y
Michigan State University	Agric. and Biosystems Eng.		(Y)[4]
North Carolina State University	Agric. and Biosystems Eng.	N	
Ohio State University	Food, Agric. and Biological Eng.	N	
Oklahoma State University	Plant and Soil Sciences	N	
University of Arizona	Agric. and Biosystems Eng.		Y
University of Arkansas	Civil Engineering	N	
University of California-Berkeley	Civil and Environmental Eng.	N	
University of California-Davis	Civil and Environmental Eng.	N	
University of Colorado	Civil, Envir. and Architectural Eng.	N	
University of Georgia	Crop and Soil Sciences	N	
University of Minnesota	Water Resource Center	N	
University of Missouri	Soil Science	N	
University of Oklahoma	Civil Eng. and Environmental Sci.	N	
University of Rhode Island	Natural Resources Sci.	N	
University of Tennessee-Knoxville	Biosystems Eng. and Soil Sci.		(Y)
University of Washington	Civil and Environmental Eng.		Y
University of Wisconsin	Agric. and Biosystems Eng.		Y

[1]The universities listed include those where a faculty member responded by the time this paper was completed to the email query of faculty at 27 universities made by the author. It is likely that most if not all universities have seminars or guest lectures available that include topics relevant to onsite and decentralized systems that students can participate in. [2]The absence of a "Y" response does not necessarily mean that another department does not offer a course, but this is viewed as unlikely. [3]For details on semester-long course offerings refer to Table 3. [4]A "(Y)" indicates that a course was offered in years past (typically for 1 to 3 deliveries) but was discontinued for varied reasons as described in the text.

Table 3. Examples of current and recent university courses focused on onsite and decentralized systems.[1]

Course information	Colorado School of Mines	University of Arizona	University of Tennessee - Knoxville	University of Washington	University of Wisconsin
Course no. and title	CEEN472 - Onsite Water Reclamation and Reuse	ABE459/559 - Design of Onsite Wastewater Treatment & Dispersal Systems	BSE532 - On-Site Domestic Wastewater Treatment, Dispersal and Reuse	CEE484 - Decentralized and Onsite Wastewater Management and Reuse	BSE372 - On-Site Wastewater Treatment and Dispersal
Course credits and delivery mode	3 cr.; classroom course delivered over 17- wk. sem.	3 cr.; online course delivered over a 17-wk. sem.	3 cr.; classroom course delivered over 17-wk. sem.	3 cr.; classroom course delivered over 11-wk qtr.	2 cr.; classroom course delivered over 10 wk. during a sem.
Delivery frequency	Every spring semester since 2006	Every even year spring semester since 2002	Fall semesters of 2006 to 2008	Quarter semester about every other year since 2007	Every fall semester since ~1999
Student level	Juniors/Seniors or Grads	Juniors/Seniors (459) or Grads (559)	Juniors/Seniors or Grads (?)	Juniors/Seniors or Grads	Juniors/Seniors
Enrollment (Avg. (range) over no. of deliveries)	18 (7 – 30)	12 (8 – 18)	6	~30	25 - 30
Course supports degree programs in:	Civil and Env. Eng.; Hydrologic Sci. and Eng.; Humanitarian Eng.	Biosystems Eng.; Agric. & Biosystems Eng.	Biosystems Eng.; Civil and Env. Eng.	Civil and Env. Eng.	Biological Systems Eng., Soils, Civil and Env. Engineering.
Pre-requisites	CEEN301 - Fundamentals of Env. Sci. & Eng. I	None required	?	CEE 357 - Environmental Engineering	Chem103 – General Chemistry I
Faculty responsible for development and initial delivery	Bob Siegrist, Ph.D. Professor, CEE	Kitt Farrell-Poe, Ph.D. Professor, ABE	John Buchanan, Ph.D., Professor Biosystems Eng. and Soil Sci.	Linda Strande Gaulke, Ph.D., Instructor, CEE	Jim Converse, Ph.D., Professor, ABE
Course delivered by other faculty (no.); current instructor	Yes (1) Bob Siegrist	Not yet Kitt Farrell-Poe	No -	Yes (1) Dave Stensel	Yes (>1) K. Karthikeyan
Source of information	Author	Poe (2014)	Buchanan (2014)	Gaulke (2014); UWCEE (2014)	Converse (2014); Karthikeyan (2004); UWCOE (2014)
Comments	Course evolved out of the CSM Small Flows Program	Course evolved, in part, out of CIDWT and its curriculum effort	Course evolved, in part, out of CIDWT and its curriculum effort	New course replaced one that had been offered earlier by R. Seabloom	Course evolved out of the UW Small Scale Waste Management Project

[1] The summary presented in this table is provided for illustration only and it may not be comprehensive with respect to all U.S. universities that offer semester-long courses focused on engineering of onsite and decentralized systems. Note: a "?" indicates that the information was not available and an "(?)" indicates what is provided is based on the author's interpretation of online course information.

Table 4. Representative comments shared by faculty involved in current or past efforts to deliver a course at their university that was focused on onsite and decentralized infrastructure.[1]

Excerpts from responses received from faculty at universities across the United States[2]
"I know of week long seminars, but nothing that is semester long… we do a lecture for civil/environmental … actually an hour and a half lecture and a 3 hour lab."
"We do not teach a semester long course. I teach one week of septic system related information in a waste water management class"
"I was able to teach a semester-long engineered-based onsite/decentralized class for three years (three deliveries). During the time I taught the course, I had grant money that … allowed me to teach. When the money ran out, I could not generate any enthusiasm among my administrators for me to continue the course."
"Our university does not offer semester-long courses in onsite systems. Students are welcome to take day long or two day short courses that are oriented towards the industry. – but usually for no credit."
"Courses of study in engineering do not leave a lot of room for electives like this."
"Very little is taught concerning the subject and what is taught is often 100 yr. old"
"Successful courses on other campuses seemed to center around a champion professor on campus."
"No, we don't have such a course and I'm not aware of other universities that do. I've toyed with the idea of developing a course focused on this topic combined with water reuse."
"We do not offer such a course and I am not aware of any other school that does. We do offer a WASH course that includes water and wastewater treatment for developing communities."
"We've wanted to do such a semester long class, but with doing 45 – 55 full and half-day classes a year for wastewater practitioners, we just haven't gotten to it"
"We do not offer a semester long course on on-site... Major course emphasis is on big pipe. Like many of my colleagues we offer a wide array of workshops and courses for practitioners."

[1] The comments listed are provided as representative remarks from faculty responding to the author's email query.

Table 5. Current course outline and work assignments for "CEEN472-Onsite Water Reclamation and Reuse", which is delivered every spring semester at the Colorado School of Mines.

Class	Topic(s)	Work assigned[1]
1	Introduction to onsite and decentralized systems	Literature reading and discussion
2	Distributed system project features and requirements	
3	Contemporary water use and wastewater generation	HW1– Estimating flow and composition and effects of efficiency and source separation
4	Water use efficiency and source separation	
5-6	Building drains and effluent sewers	HW2 – Design of small diameter gravity and pressure sewer systems
7	Guest lecture	
8-12	Septic tanks and anaerobic units, aerobic treatment units, membrane bioreactors	HW3 – Design and comparison of an ATU, MBR, and RSF
13-15	Intermittent single-pass and recirculating filters	
16	Discussion and review	
17	Exam 1	Exam 1 – coverage of material to date
18	Disinfection units	
19	Guest lecture	HW4 – Design of constructed wetlands
20-21	Constructed treatment wetlands	
22-26	Soil and landscape based treatment units	HW5 – Design of alternative soil-based treatment systems
27	Guest lecture	
28	Nutrient reduction strategies and systems	
29	Exam 2	Exam 2 – coverage of material since Exam 1
30	Systems and sustainability attributes; Performance assurance and system management	
31	Class project presentations	Project presentations (plus written report)

[1] Homework (HW) assignments and the class project are done in teams of 2 to 4 students. The exams are done individually.

Table 6. Some observations, lessons learned and recommendations based on nine years of delivery of "Onsite Water Reclamation and Reuse" at the Colorado School of Mines.[1]

Area	Observations, lessons learned and recommendations
University and department context and fit	An interested faculty member needs to consider the mission of the department and its degree programs, and how a new course focused on onsite and decentralized systems could "fit in" to areas of emphasis or concentration.
Faculty lead	An inspired faculty member needs to champion the course, develop it and get it approved, and deliver it several times to help establish it as a needed and valued course offering. A faculty member or group that has an active research program in areas relevant to onsite and decentralized systems can greatly enable development and delivery of a well-received and well-enrolled course. Early on, a 2nd or 3rd faculty member should become engaged in supporting the course and delivering it. Adjunct faculty can also contribute in an impactful manner.
Course purpose and scope	A stand-alone course can cover different aspects of onsite and decentralized systems that are targeted for the U.S. and similar industrialized nations, but also have wide applicability in the developing world. The scope should cover key topics spanning the source character and modification, treatment unit operations, and selection and design of resource-efficient recovery/reuse approaches and systems. In my view, a course should be an engineering design course and go beyond a conceptual description of approaches and technologies to one of engineering principles and processes and system selection and design. In general a course worth 3-credits is reasonable and appropriate.
Regulatory issues	System selection and design covered in the course should not be unduly constrained by current regulations and requirements, though students should be informed about the role they play in real-life projects. Students should be made aware of prescriptive vs. performance-based design.
Course title and mode of delivery	A course title should be carefully developed to reflect contemporary and future issues and concerns and to stimulate faculty and student interest in the course and its content. Depending on course purpose and scope, the mode of delivery needs to be determined. There are pros and cons to classroom versus online delivery modes.
Target audience and enrollment	A mix of juniors and seniors plus graduate students seems to work well. Students pursuing degrees in many disciplines in science and engineering can contribute to, and benefit from, an onsite and decentralized systems course. An optimum enrollment seems to be in the range of 15 to 25 students.
Pre-requisites	Students need to have an introduction to environmental science and engineering that covers the basics of environmental chemistry and biology, mass balances, water supply and water and waste treatment technologies. Due to time limitations, basic science and engineering subjects can not be covered during a course focused on engineering of onsite and decentralized systems.
Credible and rigorous delivery	The course materials and delivery should be commensurate with the intellect and creativity of the students enrolled and on par with other science and engineering courses offered at a similar level. Terminology matters and precise and consistent use is important across the different topics covered. Introducing unit process and design equations as well as modeling tools is important to convey the degree of understanding and rigor underlying the subject. Educating students on when and how to acquire necessary and appropriate information (including parameter estimation) that enables system selection and design is very important.
Guest lectures and field trips	Guest lectures by practitioners and design professionals are almost always well received by students and help reinforce material covered in class as well as enlighten students about the complexities of real-world projects. Field trips to operating systems are good but have certain logistical and timing issues.
Work requirements (and learning assessment)	Assigning design problems to be done in teams of 2 to 4 students is well received and provides a good learning experience for students. Making the design problems realistic and challenging is important but requires time and effort both to develop and grade. Using a development complex on or near campus that the students are familiar with can help elevate the students' interest in the design problems assigned and enable clear comparisons of approaches or technologies. A class project done in a team and which results in a written report and an oral presentation is well received by most students and clearly valuable to their learning.

[1] The observations and lessons learned represent the views of the author.

Nitrogen and Phosphorus Loading From Septic Systems in Small Piedmont Watersheds in North Carolina Estimated From Stream Monitoring Data

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ABSTRACT

Septic systems are being scrutinized for potential nutrient contributions to waterways. Of particular concern in North Carolina are systems in Piedmont water supply watersheds. It is important to accurately determine their actual and relative loads so that management efforts achieve truly beneficial reductions. Nutrient load estimates to date have been made using large-scale water quality models with wide ranging transport assumptions. Relevant field research to nutrient transport assessments in the Piedmont has been minimal. Stream monitoring from small Piedmont watersheds with significant septic system concentrations provides the best data source for estimating their contributions. Nutrient and flow data evaluated herein were collected from Durham, Wake, and Orange County streams in Falls and Jordan water supply watersheds by local, state and federal entities. Total nitrogen (N) and phosphorus (P) loads from undeveloped watersheds are compared to loads from watersheds with low to high septic system densities. Three contributing conditions were evaluated: base flow, stormwater flow, and storm recession flow. Data do not substantiate significant septic system base flow contributions. Relative impacts from malfunctioning systems during storm flows and from functioning systems during storm recession periods are potentially greater than from base flow. Partitioning the septic system contribution from other sources, however, is difficult. Data from small, septic-dominated watersheds should be considered in future monitoring and modeling efforts, to help advance our understanding and ability to more accurately quantify septic system nutrient contributions, and highlight where quantitative research may be most productively directed, and reduction actions focused.

Septic systems serve about 25 % and 50% of the homes in the United States and North Carolina, respectively. The potential contribution of septic systems to nutrient-impaired surface waters has received a great deal of attention in North Carolina and elsewhere. Nationally, efforts to restore the Chesapeake Bay have been in the limelight. The U.S. EPA's (2013) "Model Program for Onsite Management in the Chesapeake Bay Watershed" promotes upgrading existing systems and for jurisdictions to require enhanced N reduction for all new septic systems installed in the watershed. In North Carolina, activities have focused on comprehensive programs to restore water quality in multi-use reservoirs which provide existing or potential drinking water sources for some of the Piedmont's largest municipalities (e.g., Falls and Jordan Lakes, serving Raleigh and Cary, respectively). These Piedmont lakes have been designated by the State as "nutrient impaired," and far reaching programs mandated by a series of statutes and regulations are in various stages of implementation. Falls Lake highlights the magnitude of the challenge. About half the homes in its watershed are served by septic systems, yet nearly half of the City of Durham is also in the watershed. A considerable amount of agricultural land also drains into the lake, so there are plenty of nutrient sources as possible candidates for mitigation efforts.

In the past, modeling has been used to quantify and partition the relative contribution of septic systems in nutrient-sensitive watersheds, nationally and within North Carolina. The models use similar estimates of nutrient loads delivered into the ground from a "typical conventional" septic tank. The transport assumptions then applied vary more widely, with little justification to either support or refute them. For example, in the Chesapeake Bay model, it is assumed that 40% of the N leaving the "edge" of a septic system gets into the Bay (EPA, 2013). The watershed analysis risk management framework (WARMF) model has been used to estimate septic system contributions to Falls Lake (DENR, 2010). Results indicated they contribute 9% of the P and 14% of the N loading to the lake, with 7% of the septic system-generated P and 14% of the septic system-generated N estimated (by the model) to reach surface waters. The NC Division of Environmental Health found these estimates to be unreasonably high (DEH, 2010), while a contractor for the city of Raleigh argues that they could be substantially understated, depending on the modeling assumptions made (Hazen and Sawyer, 2013).

Field-based studies of nutrient transport down-gradient from septic systems in the Coastal Plain of North Carolina have been reported (Humphrey et al., 2010; Humphrey et al., 2013; Pradhan, 2004). Even where groundwater transport rates are found to be high in sandy coastal soils, attenuation has been shown to be rapid, especially when there are organic or phreatic zones intersecting the flow path to adjoining waterways. Transport mechanisms and expected performance in Piedmont soils have been less studied. Work at the University of Georgia (both field experiments and HYDRUS II modeling) has demonstrated a high potential for denitrification within one meter of the trench infiltrative surface in soils similar to those found in the North Carolina Piedmont (Bradshaw and Radcliffe, 2011; WERF, 2010).

MATERIALS AND METHODS

A number of monitoring programs have collected relevant data for evaluating septic system impacts in North Carolina's Piedmont. Locations and details of the sites where data presented in this paper were collected are shown in Fig. 1 and Table 1, including web-links to the data where applicable. Monitoring has been undertaken at these sites for a variety of purposes. The Flat River Tributary site was a long-term United States Geological Survey (USGS) ambient monitoring station (discontinued in 2012). Data on its nutrient mass loads are included in a comprehensive USGS report (Harden et al., 2013). Hill Forest is a North Carolina State University (NCSU) Demonstration Forest within the Flat River Watershed. Research here includes evaluations of forest management practices on stream water quantity and quality (Boggs et al., 2013). Four sites (Duke Forest, Crooked Creek Tributary, Seven Mile Creek Tributary, and Cabin Branch Tributary) have been monitored as part of two USGS studies on the effects of wastewater treatment practices on the occurrence of selected traditional and emerging contaminants in streams draining small unsewered and sewered watersheds (Ferrell and Grimes, 2014; Ferrell et al., 2014; Ferrell, 2011). The Wake County sites have been monitored by NCSU's Water Quality Group and Wake County's Stormwater Programs, supported by grants from the State's 319 program (Line, 2010; Line, 2013; Hobby, 2012). Collins Creek is the background site for a USGS study on the effects of land application of wastewater biosolids on water quality in the Jordan Lake watershed (Wagner, 2012). The larger Cabin Branch site was part of another USGS study of N sources in three small watersheds with varying levels of urbanization (McSwain et al., 2013).

Monitoring data from these sites are reported and compared. Flow data are used to calculate delivered loads from the watersheds. Most of the data presented are of loads during base-flow conditions. Comparative loads during storm events are also presented where these could also be determined. For sites containing low and medium to high densities of septic systems, the projected contribution of septic systems to base-flow loading is presented, assuming the entire nutrient loading during base-flow conditions is septic-generated.

RESULTS AND DISCUSSION

Background (Undeveloped) Sites: Data from three undeveloped sites in the region serving as "background" watersheds are presented in Table 2. These data reflect the expected natural variability of water and nutrient concentrations and mass loads from undisturbed, primarily forested watersheds. Factors contributing to the variability include soil variations, differing climatic conditions during sampling events and watershed size. Note that only the Flat River Tributary data, and the Hill Forest data are based on complete datasets for their respective monitoring periods. The "base" and "storm" data from Flat River Tributary are from days when grab-samples were collected (approximately monthly). The Duke Forest results are averages from five grab samples collected during the monitoring period.

A relatively small percentage of the N at these background sites is in the nitrate/nitrite form (3 to 16%), with the majority being in the organic form. Comparing the extensive Flat River Tributary "Complete," "Storm," and "Base" data sets shows that storm events contribute the majority of the net water and nutrient loads, even in the background watersheds.

Low Density Septic Sites: Data from two sites with a relatively low density of septic systems are summarized in Table 3. The data from both sites are derived from monthly grab samples and flow readings. Flows for Beaverdam, 2010-12, were estimated from the relationship established in 2008-09 between gage readings and discharge measurements (gage readings only were taken during the 2010-12 monitoring period). These data show minimum differences in yields, compared to the background stations. The Collins Creek storm data is comparable to the Flat River (background) storm data yield, though with a higher proportion of the total N in the nitrate/nitrite form (16% of the N is in nitrate/nitrite form in Collins Creek *vs*. 3% in Flat River). The Beaverdam 2010-12 data also shows a higher percentage of N in the nitrate/nitrite form, compared to the 2008-09 period (31% *vs*. 6%), though the net mass nutrient load remains low. The 2010-12 period was generally wetter than in 2008-09.

Medium to High Density Septic Sites: Data from six sites with a medium to high septic system density are summarized in Table 4. Data were derived from monthly grab samples and flow readings. Flows for Honeycut and Cedar for 2010-12 were estimated from the relationship established in 2008-09 between gage readings and discharge measurements (gage readings only were taken during 2010-12). Comparisons of nutrient loading from the monitored watersheds, ordered by increasing septic system density, are depicted in Figs. 2, 3, and 4. Mass nutrient loads are similar to those from the background sites, with modest indications of increased nutrient loads as the density of septic systems increases. The watershed evaluated with the highest system density (Cabin Branch Tributary) had an N loading lower than from the background Duke Forest site concurrently sampled, and had only a slightly higher ortho-phosphate loading. A higher percentage of the total N mass loads from these septic-dominated watersheds is in the nitrate/nitrate form, compared to the background sites. Data from the generally wetter 2010-12 periods at Honeycut and Cedar also indicate higher loadings compared to 2008-09.

Watershed mass nutrient load measurements during base flow conditions are also compared to estimates of total mass loads generated by the septic systems. In Table 5, accepted values for N and P in septic tank effluent of 5.0 and 0.82 kg per capita per year, respectively, are used to calculate the maximum percentage of measured loads potentially delivered by the on-site systems in the watersheds (Tetra Tech, 2013; Lowe et al., 2009). Delivery rates were found to range from 1.6 to 8.4% for N and 1.0 to 8.5% for P. These are likely overestimates, as they assume 100% of the measured stream loads are derived from the on-site systems, and do not make any adjustment for expected background levels or take into account other sources (e.g., natural soils, pets, atmospheric deposition, lawn fertilizers, etc.).

Septic Contributions During Storm and After-Storm Recession Flows: Stormwater contributions potentially far outweigh base-flow contributions, both from undeveloped and septic system-populated watersheds (see Flat River Tributary and Collins Creek data, Tables 2 and 3). Collins Creek data allows further quantification of relative storm-flow contributions. Flow values were estimated for 492 days during the monitoring period, including 126 “storm event days” and 366 “base-flow” days. This partitioning was determined using the daily flow hydrograph derived from the flow-stage relationship between flow-stage readings reported during the sampling days and applying this relationship to the USGS gage data reported for the 492-day period. Average base-flow day discharge was 4.08 l/s and average concentrations were 0.64 mg N/L and 0.043 mg P/L, yielding base-flow loads of 82.5 and 5.5 kilograms N and P, respectively, for the 366 “base-flow” days. Average storm-flow day discharge was 80.4 l/s and average concentrations were 1.51 mg N/L and 0.23 mg P/L, yielding storm-flow loads of 1322 and 201 kilograms N and P, respectively, for the 126 “storm-event” days. For the 492-day period, “storm-events” thus are estimated to have contributed 94% of the N and 97% of the P from this watershed.

Two USGS studies cited herein further allow assessment of the relative septic system contribution during base flow, storm flows, and the storm-water recession period. Potential septic system loads are expected to be greater during storm flows from surface-malfunctioning systems, and from subsurface (non-surfacing) systems during storm recession periods. The recession period occurs between the peak rainfall-induced hydrograph (which includes both surface runoff and groundwater flow) and when base-flow (ground-water only) again dominates the stream flow. A properly-functioning system could contribute to a greater degree during this period if storm-recharged interflow and groundwater beneath the system flushes through the watershed and conveys to streams a higher than normal contribution from the septic systems.

Ferrell (2011) resampled the Duke Forest (background) site and three septic-dominated watersheds in Durham County during four separate storm recession periods in 2012-13. These were the same sites sampled in 2004-05 during base flow periods (Ferrell and Grimes, 2014; Ferrell et al., 2014). No storm-related response at the Duke Forest (background) site was observed during two monitored storms. Storm recession results from the three septic system-dominated watersheds each show a recession curve spike in nitrate/nitrite N (for example, see Fig. 6). No storm-related response in ortho-phosphate concentrations was observed.

Another USGS study completed by the USGS included isotopic analysis of nitrate in stream samples collected from Cabin Creek during eight base-flow days and three storm-flow days (McSwain et al., 2013). The primary source was determined to be soil nitrogen, with a smaller number of samples showing a mixture of nitrate derived from precipitation and soil, and one sample indicating the source to be fertilizer. No evidence of a septic system source was found.

CONCLUSIONS

Base flow sampling is a reasonable approach to evaluate the impact of “functioning” (non-surfacing) septic systems on nutrient loads from small Piedmont watersheds. Data reported herein from small watersheds in North Carolina, even from high septic system density watersheds, do not substantiate a significant contribution of N and P attributable to septic systems during base flow conditions. Little nutrient loading benefit to impaired surface waters would appear to be gained by upgrading existing functioning (non-surfacing) systems, or by requiring new systems to utilize enhanced nutrient reduction technologies where conventional technologies would otherwise be acceptable.

A more significant impact noted from the higher density sites is the increase in both the nitrate/nitrite loading and in the proportion of the N in the nitrate/nitrite form. There is little evidence, however, that higher septic system density is the primary contributing factor to the observed shift to higher nitrate levels in the residential watersheds.

Determining the relative contribution of septic systems to stormwater and storm recession loadings is more difficult and where future efforts should be focused. The relative importance to total nutrient loading of higher-flow periods was clearly evident from data collected at a background site (Flat River Tributary) and a low-density septic system site (Collins Creek). In higher-density septic system basins, malfunctioning systems are expected to have the greatest potential to contribute loading during storm events. We are aware of no direct measurements to quantify such contributions. However, finding and repairing surface-failing systems would likely have a significant impact on reducing potential septic system contribution.

The reported storm recession nutrient "surge" indicates this may be a contributing path of greater significance from "properly functioning" systems, but which also remains elusive to quantify. Isotope analysis during storm-event sampling at Cabin Creek showed no "septic" signature (McSwain et al., 2013**)**. Ferrell's storm recession event samples from three septic-dominated watersheds may shed additional light on this, when isotope analyses of these have been completed (Ferrell, 2011).

Continued attention should be given to the potential impacts during storm periods from surface-malfunctioning systems and from properly functioning systems during storm recession conditions. Data from smaller septic-dominated watersheds should be considered for future monitoring and modeling efforts, potentially yielding more meaningful and justifiable results on the role septic systems truly play, as well as to assess the benefits of mitigation options.

Acknowledgements: A number of individuals made invaluable contributions enabling the author to assemble the information presented in this paper. These include Steve Bristow and Kathryn Hobby, Wake County Environmental Services; Bob Jordan and Matthew Yearout, Durham County Health Department; Gloria Ferrell and Kristen McSwain, USGS; Johnny Boggs and Daniel Line, North Carolina State University; and Vic D'Amato, Tetra Tech. Their contributions are greatly appreciated.

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Table 1. Small piedmont watershed monitoring sites in Durham, Orange and Wake Counties, NC

Site (No. refers to location indicated on Fig. 1)	USGS ID	County (major river basin)	Watershed area (ha)	Population on on-site, no. (density, Pop/ha)†	Data collector (reported period)
Background (undeveloped) sites					
1. Flat River Tributary	0208650112	Durham (Neuse)	295	13 (.04)	USGS (1997-2012)
2. Hill Forest		Durham (Neuse)	29.0 (#1) 39.9 (#2)	0	NCSU (2008-2010
3. Duke Forest (Rhodes Creek Tributary)	360138078592101	Orange (Neuse)	49.2	0	USGS (2004-05, 2008, 2012-13)
Low density septic system sites					
4. Beaverdam Creek		Wake (Neuse)	627	264 (0.42)	NCSU and Wake Cty (2008-12)
5. Collins Creek	0209691590	Orange (Cape Fear-Jordan Lk)	427	205 (0.48)	USGS (2011-2013)
Medium to high density septic system sites					
6. Honeycut Creek		Wake (Neuse)	420	718 (1.71)	NCSU and Wake Cty (2008-12)
7. Cabin Branch	0208525105	Durham (Neuse)	894	1555 ST, 287 SF (2.06)	USGS (2011-2012)
8. Crooked Creek Tributary (Green Briar)	360543078552401	Durham (Neuse)	161	406 ST, 36 SF, 14 UK (2.84)	USGS (2004-05, 2008, 2012-13)
9. Cedar Creek		Wake (Neuse)	389	1271 (3.27)	NCSU and Wake Cty (2008-12)
10.Seven Mile Creek Tributary (Inverness)	360501078580201	Durham (Neuse)	33.7	127 St, 7 UK (3.98)	USGS (2004-05, 2008, 2012-13)
11. Cabin Branch Tributary (Paragon)	360609078530901	Durham (Neuse)	38.9	137 ST, 12 SF, 31UK (4.63)	USGS (2004-05, 2008, 2012-13)

†Population data based on housing counts in watersheds provided by Durham, Wake and Orange County GIS Mapping Programs, representing conditions approximately in 2010. Population per household (2.4 people/home) based on 2000 Census data for Wake County.

Table 2. Monitoring results for background (undeveloped) watersheds								
Site		**Monitoring period**	**Type of data**	**Water yield (M^3/yr/ha)**	**TN**	**NOx**	**TP**	**OrthoP**
					kg/ha/yr			
Flat River Tributary		1997-2008	Complete	2008	0.84	0.091	0.088	
		2009-2012	Base	678	0.154	0.019	0.016	
			Storm	13760	12.5	0.378	1.78	
Hill Forest	**W1**	2009 WY (4/08-3/09)	Complete	1520	0.666	0.032	0.081	
	W2		Complete	1505	0.946	0.081	0.189	
Duke Forest		2004-05	Base	1520	0.455	0.072		0.030

Table 3. Monitoring results from low-density septic system watersheds						
Site	**Monitoring period**	**Type of data**	**Water yield (M^3/yr/ha)**	**TN**	**NOx**	**TP**
				kg/ha/yr		
Beaverdam	2008-09	Base	525	0.126	0.007	0015
	2010-12	Base	577	0.178	0.053	0.029
Collins Creek	3/11-4/13	Base	307	0.201	0.034	0.014
		Storm	18680	27.5	4.34	4.66

Table 4. Monitoring results from medium to high density septic system watersheds							
Site	**Monitoring period**	**Type of data**	**Water yield (M^3/yr/ha)**	**TN**	**NOx**	**TP**	**OrthoP**
				kg/ha/yr			
Honeycut	2008-09	Base	574	0.213	0.087	0.016	
	2010-12	Base	783	0.406	0.203	0.043	
Cabin	7/11-6/12	Base	558	1.42	0.039	0.01139	
Crooked	2004-05	Base	746	1.00	0.687		0.019
Cedar	2008-09	Base	482	0.420	0.038	0.025	
	2010-12	Base	512	0.403	0.0438	0.030	
Seven Mile	2004-05	Base	489	0.631	0.161		0.017
Cabin (Paragon)	2004-05	Base	482	0.290	0.0359		0.010

Table 5. Estimated maximum potential percentage of septic-generated nutrients contributing to steam base-flow nutrient levels.								
Site		**Population on on-site system (no. on septic or sand filter)**	**On-site generated nutrients (kg/yr)**		**Measured load in stream (kg/yr)**		**Delivered to stream (Max. % on-site generated)**	
			N	**P**	**N**	**P**	**N**	**P**
Beaverdam Creek	**2008-09**	264	1318	216	79.1	9.67	6.0	4.5
	2010-12	264	1318	216	105.5	18.2	8.0	8.5
Collins Creek		205	1024	168	86.2	5.84	8.4	3.5
Honeycut Creek	**2008-09**	718	3586	587	89.6	6.90	2.5	1.2
	2010-12	718	3586	587	171	17.9	4.8	3.1
Cabin Branch		1683	8405	1375	326	34.5	3.9	2.5
Crooked Creek Tributary		456	2277		161		7.1	
Cedar Creek	**2008-09**	1271	6347	1039	163	9.90	2.6	1.0
	2010-12	1271	6347	1039	157	11.76	2.5	1.1
Seven Mile Creek Tributary		134	669		21.3		3.2	
Cabin Branch Tributary		180	899		11.3		1.3	

Figure 1. Locations of watershed monitoring sites in Durham, Orange and Wake Counties in the Falls and Jordan Lake watersheds, NC (see Table 1 for site names, details and data links).

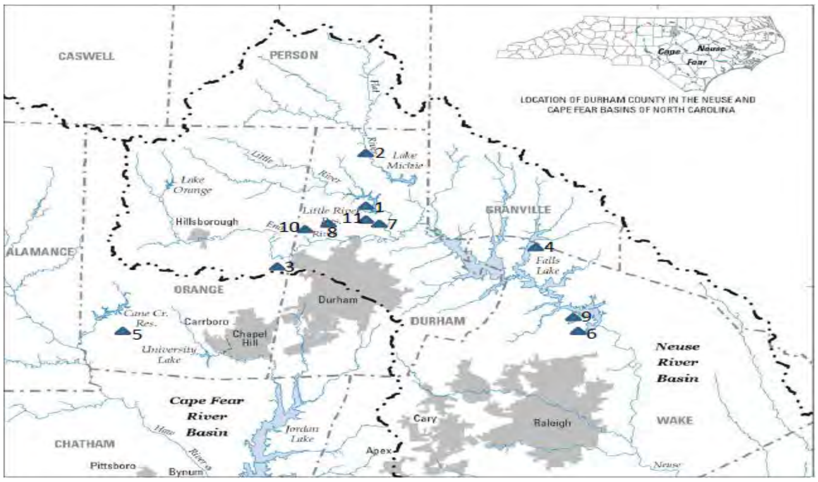


Figure 2. Total nitrogen loading from small Piedmont watersheds monitored in North Carolina.

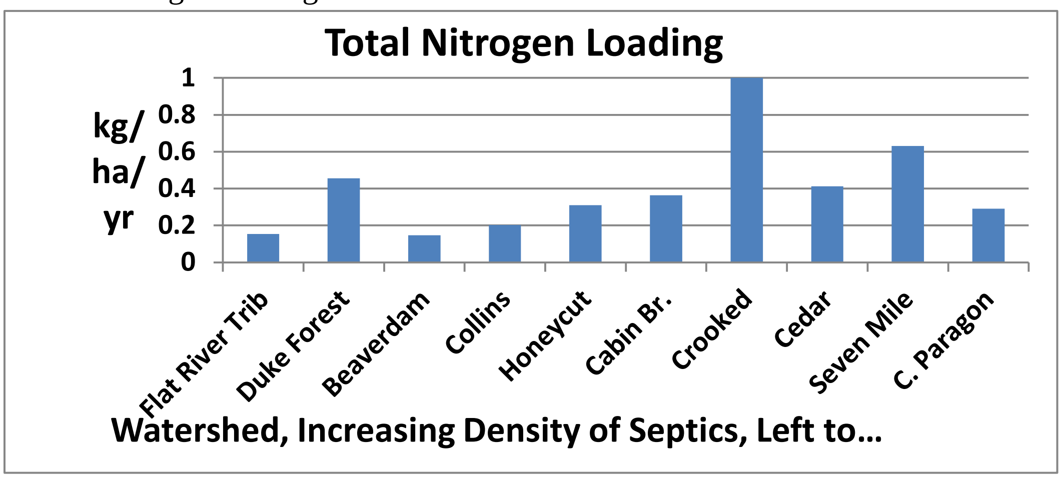


Figure 3. Total phosphorus loading from small Piedmont watersheds monitored in North Carolina.

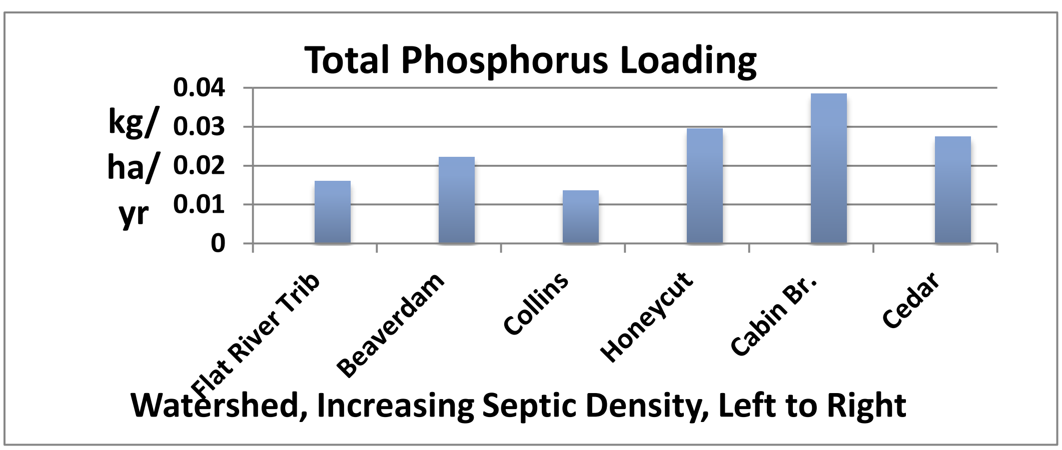


Figure 4. Percent nitrate+nitrite of total nitrogen in monitored watersheds.

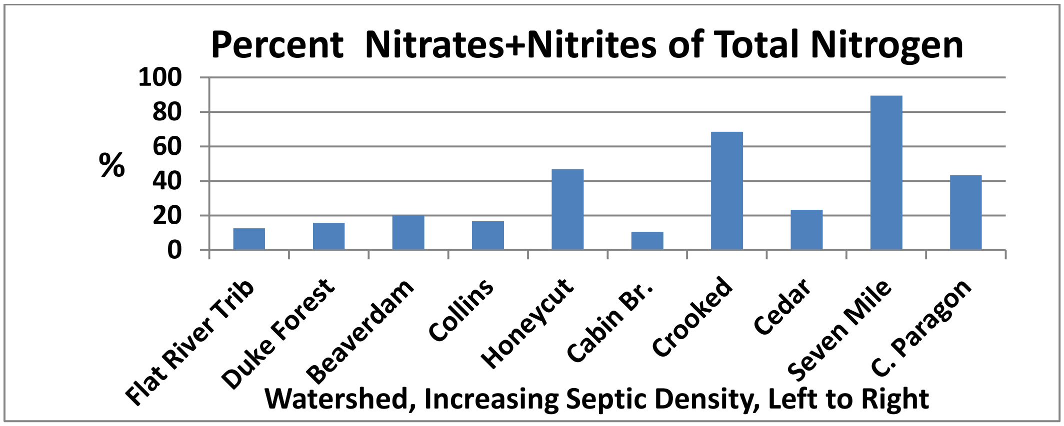
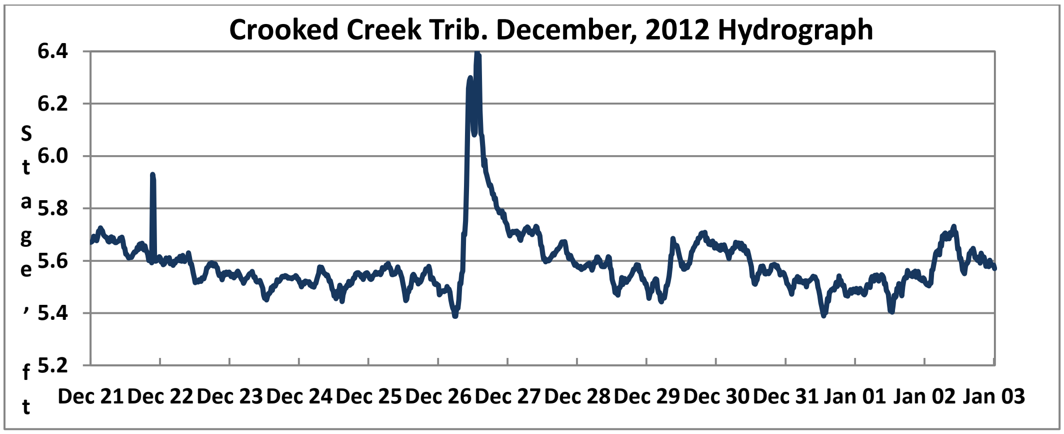
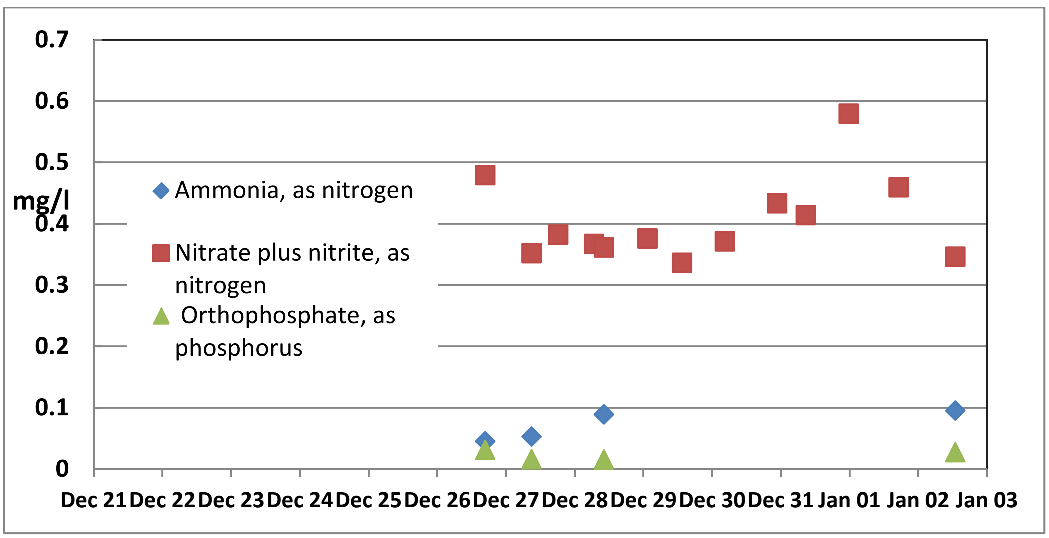


Figure 7. Nutrient variations during stormwater recession hydrograph, Crooked Cr. Trib.





Impact of onsite wastewater treatment systems on nitrogen and baseflow in urban watersheds of metropolitan Atlanta

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ABSTRACT

Onsite wastewater treatment systems (OWTS) are widely used in the southeastern United States. OWTS can be a source of nitrogen pollution of surface and ground waters as a result of poor maintenance or high density. In this area most of the public water supply comes from surface water withdrawal. So, the impact of OWTS on surface water quality and quantity must be investigated. The main goal of this project is to demonstrate the impact of OWTS on the N load and the base-flow in urbanizing watersheds of Metropolitan Atlanta, Georgia. Synoptic samples of 24 watersheds were taken 3 times per year for 2 years and stream discharge was measured, both under base-flow conditions to consider seasonal variations. Preliminary results of differences in N load and base-flow, and other water quality parameters such as electrical conductivity (EC) and chloride (Cl^-) in streams impacted by watersheds with high and low density OWTS are presented. Base-flow yields were not significantly different in various seasons except in the summer 2012 (*p-value* <0.001). Nitrate concentration increased linearly with OWTS density beyond a threshold of 100 OWTS per square kilometer (R^2=0.671). In watersheds with low density septic system, some streams with high concentration of nitrate were also observed. Analysis of $\delta^{15}N$ values versus nitrate concentration revealed variation of $\delta^{15}N$ values mainly due to the mixing of two or more sources of nitrate rather than microbial denitrification. EC and Cl^- concentrations also increased with OWTS density. Further analysis is needed to determine the impact of OWTS on water quality and quantity at the watershed scale.

With urbanization, a major pollution issue arises in the form of human waste disposal (Mallin, 2009). From 2000 to 2010 the population of the four-state region consisting of Alabama, Georgia, South Carolina and North Carolina increased 7.5 to 18.5% with an 18.3% increase in Georgia (U.S.Census, 2010) More than 65% of public water supply in these four regions is provided by surface water due to low yielding wells caused by fractured bedrock (Clarke and Peck, 1991, Fanning, 2001).

In 2007, approximately 25% of the total housing units in the United State used OWTS which released about 15 billion liters of effluent per day(Oakley et al., 2010). Approximately 46% of the homes with OWTS were located in the Southern region of the United States(U.S.EPA, 2008). Approximately 30% of the homes in Georgia are on OWTS. In 2005, the number of OWTS in Metropolitan Atlanta was estimated to be approximately 526,000 (MNGWPD, 2006) and the number is expected to increase with population growth in this region. Due to the high costs of centralized systems, OWTS is no longer considered a temporary solution to be replaced eventually by centralized collection and treatment (U.S.EPA, 2002). However, the design of conventional OWTS is not particularly effective for N removal. It has been estimated that N loading is reduced by only 10–20% before discharge to the environment (Oakley et al., 2010).

Failing or high density OWTS can cause poor water quality in both surface water and groundwater (Conn et al., 2012). Nutrients are the third most common cause of river and stream impairment, and the second most common reason of impairment in lakes, reservoirs, and ponds listed on the EPA 303(d) list (U.S.EPA, 2012). Nitrate is a concern in surface and ground waters that serve as drinking water sources because it can cause methemoglobinemia or blue-baby syndrome (Beal et al., 2005). The maximum nitrate contaminant level for drinking water to protect against methemoglobinemia is set to 10 mg/L by U.S.EPA (Kaushal et al., 2006). Although nitrogen is critical to ecological health, excessive loading of this element causes eutrophication and hypoxia in marine and brackish water ecosystems (Conley, Paerl, et al., 2009, Pinckney, Paerl, et al., 2001, Rabalais, 2002) and disrupts the aquatic food web in freshwater streams (Benstead et al., 2009; Davis et al., 2010; Gulis et al., 2004; Suberkropp et al., 2010). Threshold concentrations for N in water bodies are quite low and depend on the type of the water resource (U.S.EPA, 2002). Only a couple of studies have confirmed the source of nitrate to be from OWTS by using source tracking techniques in ground water systems (Aravena et al., 1993, Aravena and Robertson, 1998). Landers and Ankcorn (2008) found that watersheds with high density OWTS had higher base-flow compared with watersheds with low density OWTS. They concluded that although expanding urbanization can accelerate transport of runoff into streams due to an increase in impervious areas, OWTS can increase stream base-flow (Landers and Ankcorn, 2008).

The objective of this study was to determine the impact of OWTS on the N load and base-flow in urbanizing watersheds of Metropolitan Atlanta, Georgia. This paper presents preliminary results on N, base-flow, and other water quality parameters in streams impacted by watersheds with high and low density OWTS.

MATERIALS AND METHODS

The study area has been described in detail in Landers and Ankcorn (2008). The area is in the Southern Piedmont region, southeast of Atlanta, GA and has a mean annual precipitation of about 128 cm(National Weather Service, 2007). The selected watersheds, ranged in area from 0.181 to 8.8 km^2 (average area 2.5 km^2). Out of the 24 selected watersheds, 12 are characterized as having high density of OWTS (HDS) with the remaining twelve characterized as having a low density of OWTS. We divided the watersheds into two groups: watersheds with less than 50 OWTS per square kilometer were considered LDS and watersheds with more than 50 OWTS per square kilometer were considered HDS. Other watershed selection criteria included similar geological setting, precipitation, climate, accurate base-flow measurement locations and available spatial datasets of natural, infrastructure, and water-use characteristics.

Synoptic measurements of base-flow were taken simultaneously with water sampling three times per year for 2 years to obtain the seasonal flow variations. Stream discharge measurement for the 24 sites were conducted using the current-meter method (Rantz, 1982).Water samples were collected during base-flow periods 3 times per year from 24 sampling stations at the outlets of these watersheds at the same time as synoptic streamflow measurements. Periods of base-flow suitable for synoptic measurement were determined by examining streamflow from those sites equipped with stream gages. Appropriate sampling periods usually occur following stabilization of flow rates after a storm event, or during periods without rainfall. Basic water quality parameters such as temperature, pH, electrical conductivity, and dissolved oxygen were measured using multi-parameter water quality probes. All samples were analyzed by the University of Georgia Environmental Services Laboratory for NH_4^+, NO_3^-, total Kjeldahl nitrogen (TKN), $\delta^{15}N$, and Cl^-. All statistical analyses were conducted with level of significance of α=0.05 in SAS software.

Chloride (Cl^-) served as a conservative tracer to observe N transformation and the effect of dilution within the watersheds. The isotope ^{15}N in nitrate was used to distinguish different sources of N due to the preference of biological organisms for the lighter isotope of nitrogen (^{14}N). Denitrification can increase $\delta^{15}N$ values as nitrate concentration decreases, while mixing of nitrate from two or more sources can cause both $\delta^{15}N$ and nitrate concentration to increase (Mayer, Boyer, et al., 2002). The molar ratio of ^{15}N to ^{14}N has been used to characterize N from different origins, including human waste and animal waste (+7.6 to 25 ‰), and commercial fertilizer (-2 to +4 ‰).

RESULTS AND DISCUSSIONS

Synoptic samples and discharge measurements of 24 watersheds were taken 6 times (in November 2011, March 2012, July 2012, November 2012, April 2013, and July 2013). Average base-flow yield in the 6 measurements were not significantly different except for July 2012 (*p-value* < 0.001) (Fig.1). In a study of watersheds in the Metro Atlanta area, Calhoun et al. (2003), showed that base-flow yield decreased in watersheds with higher percentage of imperviousness due to more surface runoff and less infiltration contributing to deep percolation and base-flow. In our study, impervious surface percentage increased with OWTS density. Since our results show the same or higher base-flow yield within the HDS watersheds, this implies OWTS effluent may have offset the effect of impervious surface.

Figure 2(a) shows nitrate concentrations as a function of OWTS density in 24 watersheds for all six sampling events. NO_3^- concentrations decreased linearly with OWTS density within the LDS watersheds (R^2=0.26), and increased linearly with OWTS density within the HDS watersheds (R^2=0.64). The high nitrate concentrations in some LDS watersheds implied other sources than OWTS such as agricultural runoff, leaking sewer lines, or livestock/wildlife.

Figure 2(b) shows $\delta^{15}N$ as a function of OWTS density within the watersheds. Regression analysis of $\delta^{15}N$ values in the LDS watersheds revealed a linear decrease with OWTS density (R^2=0.26) and in the HDS watersheds a linear increase with OWTS density (R^2=0.53). $\delta^{15}N$ values for HDS watersheds with high nitrate concentrations were in the range reported for human and animal wastes . Some LDS watersheds also had $\delta^{15}N$ in this range. These results suggest that human wastewater from septic systems was the source of N in high density watersheds and that human (from leaking sewer lines) or animal (from livestock or wildlife) were the source of N in the low density watersheds.

$\delta^{15}N$ values increased as a function of nitrate concentrations with a 2nd degree polynomial regression in both HDS and LDS watersheds with R^2 of 0.65 and 0.86, respectively (Fig. 3). This suggests that the high $\delta^{15}N$ values can be primarily attributed to mixing of two or more sources of nitrate with different values rather than microbial denitrification of a source with low $\delta^{15}N$ values. Chloride concentrations and electrical conductivity as a function of OWTS density within the watersheds are shown in Figure 2 (c and d). Regression analysis showed a linear increase in Cl^- concentrations (R^2= 0.38) and EC (R^2=0.53) with OWTS density within the watersheds.

Chloride can derive from other sources such as agricultural runoff, so more research is needed to identify the dominant sources of Cl^- within the watersheds.

CONCLUSION

Base-flow yield results indicated the same or higher discharge in the HDS watersheds due to the presence of OWTS effluent which may off-set the effect of impervious surface. Analyses of nitrate concentrations of six synoptic measurements revealed significant linear relationships with OWTS density: a linear decrease below the threshold of about 100 OWTS per square kilometer and linear increase above this threshold. High nitrate concentrations in LDS watersheds were attributed to some probable sources other than OWTS such as runoff from agricultural landuse, sewer line leakage or livestock/wildlife. Since $\delta^{15}N$ values were positively correlated with nitrate concentrations, it can be concluded that mixing of two or more sources of nitrate with different $\delta^{15}N$ values was responsible for the high values, not denitrification of a source with low $\delta^{15}N$ values. Increasing EC and Cl^- concentrations with OWTS density also suggested the presence of OWTS effluent in streams. Further analysis is needed to distinguish other sources of nitrate than OWTS within the LDS watersheds, and to determine the impact of OWTSs on water quality and quantity at the watershed scale.

AKNOWLEDGEMENTS

This research was funded by a grand from the USDA-NIFA National Integrated Water Quality Program.

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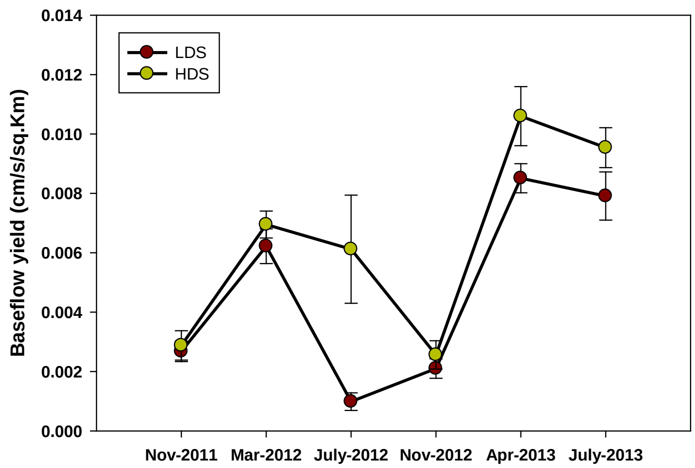

Figure 1. Base-flow yield in streams of watersheds with low density (LD) and high density (HD) of OWTS in November 2011, March 2012, July 2012, November 2012, April 2013, and July 2013.

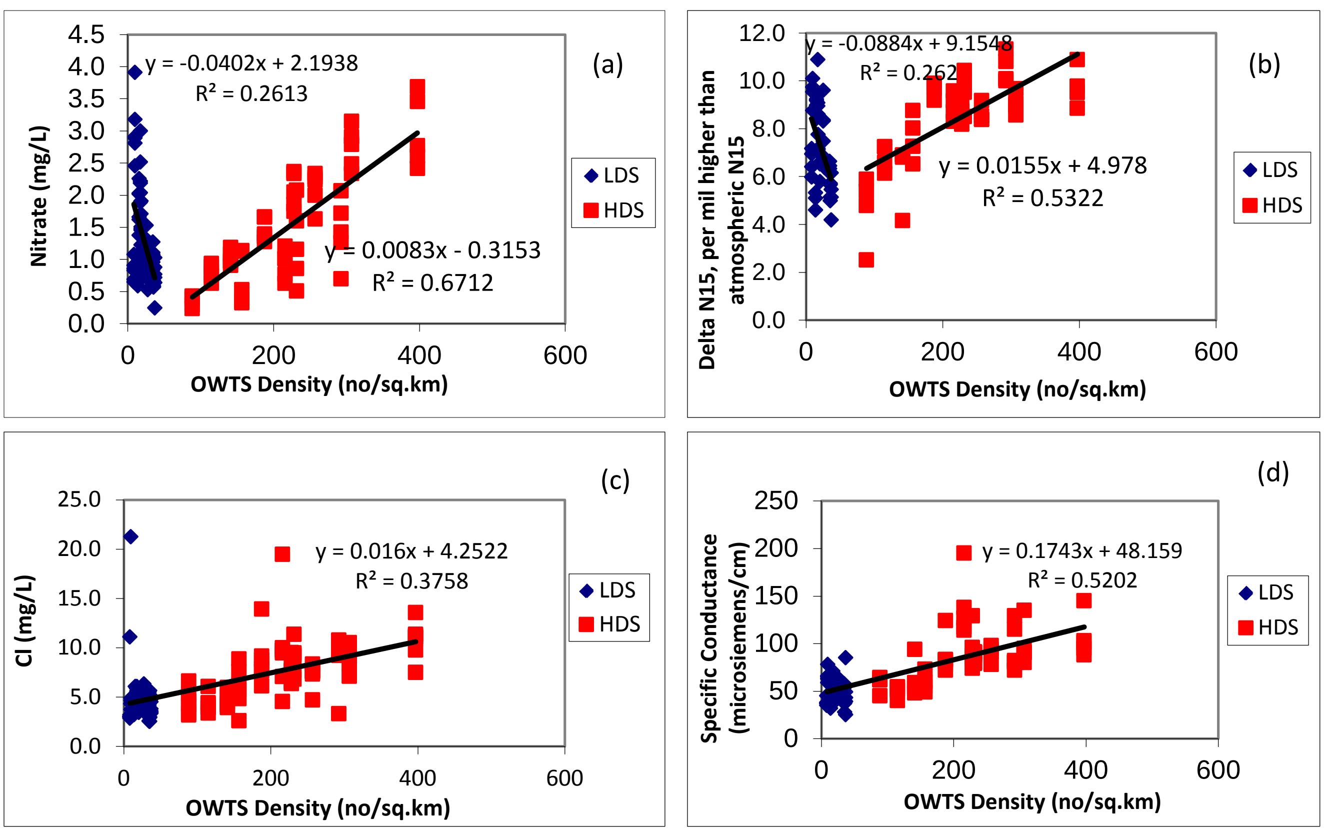

Figure 2. Nitrate concentration (mg/L) (a) , $\delta^{15}N$ value (‰) (b), chloride concentration (mg/L), and electrical conductivity (μS/cm) as a function of on-site wastewater treatment system (OWTS) density within the watersheds in November 2011, March 2012, July 2012, November 2012, April 2013, and July 2013.

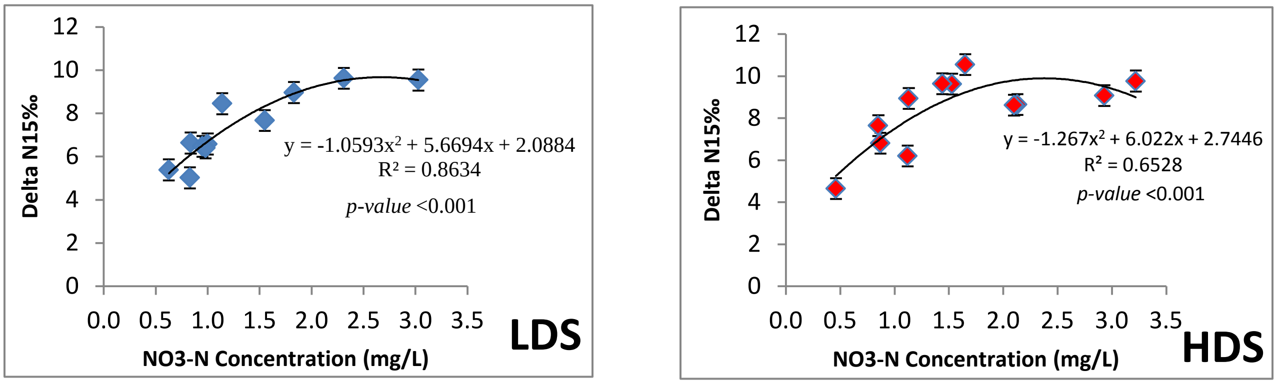

Figure 3. Mean of $\delta^{15}N$ value (‰) vs. nitrate concentration (mg/L).

Paired Watersheds Approach For Evaluating The Influence Of Wastewater Management Strategies On Stream Nutrient Concentrations.

Charles Humphrey, East Carolina University

ABSTRACT

Excess nutrients in North Carolina surface waters have resulted in water quality impairment and aquatic habitat degradation. A better understanding of the watershed-scale influences of various wastewater treatment methods is needed for the development of comprehensive watershed nutrient management regulations. The goals of this study were to determine if significant differences in total nitrogen (TN) and total phosphorus (TP) concentrations and loads were observed in watersheds served by onsite wastewater systems (OWS) in relation to watersheds served by municipal sewer (MWS) and to determine if there were significant differences in nutrient treatment efficiencies between OWS and MWS. Ten sites (5 OWS and 5 MWS) in Pitt County, NC were instrumented with piezometers and sampled seasonally (4 times over 1 year) for groundwater nutrient analysis and physical and chemical characterization of groundwater and wastewater. Influent and effluent samples from the MWS were also collected and analyzed for TN and TP analysis to determine nutrient reduction efficiency. Seven streams (3 OWS and 4 MWS) were monitored monthly for flow, nutrient concentrations, and physical and chemical parameters for one year (August 2011-August 2012). Groundwater and stream TN and TP concentrations and loads in watersheds served by OWS were higher than groundwater and stream nutrient concentrations and loads in MWS watersheds. However, the 4 MWS streams that were monitored did not receive MWS treatment plant effluent. The TN and TP treatment efficiencies for the monitored OWS were greater than or equal to municipal sewer treatment plant efficiency. Because OWS, like MWS, influence stream TN and TP concentrations and loads, OWS contributions should be considered when nutrient management strategies for watersheds are developed.

Impact of onsite wastewater treatment systems on stream fecal bacteria: Case study of urbanizing watersheds in metropolitan Atlanta, Georgia

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ABSTRACT

Onsite wastewater treatment systems (OWTS) are an integral part of the wastewater infrastructure in the United States. Thus the effectiveness of these systems in removing contaminants from wastewater cannot be overemphasized. At the watershed scale, the impacts of onsite systems as a prominent source of fecal pollution to groundwater and surface waters have not been adequately elucidated. Water quality monitoring can provide the tools needed to understand the spatial and temporal dynamics of bacterial yield as impacted by onsite wastewater systems. Our study assessed the influence of OWTS on the microbial quality of water in streams in watersheds impacted by high or low density of OWTS. The seasonal and temporal distribution of fecal bacterial yields in streams was evaluated and the correlation with OWTS density and specific conductance were examined across 24 well characterized watersheds ranging in area from 18 to 880 ha (0.07 to 3.4 mi^2). The selected watersheds are in the Ocmulgee and Oconee River basins in the Southern Appalachians region – a sensitive ecological zone with 65% of rivers and streams in poor condition. Our data suggests a positive correlation between microbial water quality parameters and OWTS density exceeding 88 units/km^2 (229 units/mi^2). This relationship is statistically significant during the summer season of low streamflow, indicating the seasonal dependence of bacterial loading as impacted by OWTS density. Ongoing work entails fecal source tracking and watershed modeling to complement routine indicator bacteria monitoring.

INTRODUCTION

Onsite wastewater treatment systems, popularly known as septic systems, are an integral part of the wastewater infrastructure in the United States (US EPA, 2002). The US Census Bureau (2009) reports that approximately 24% of all housing units in the United States rely on onsite wastewater systems for wastewater treatment. This percentage is set to increase in response to rapid population growth and the growing housing market in peri-urban communities. Moreover, improved knowledge of OWTS functioning, management options and technology have combined to improve public perceptions and provide a favorable regulatory environment for the utilization of onsite wastewater systems. US EPA (2002) estimates that ~33% of all new housing units use some form of OWTS, a reflection of the growing confidence in the effectiveness and the long-term viability of the decentralized approach to wastewater treatment.

Recent reports of widespread water quality impairments with fecal pathogens have raised serious questions about the sources of fecal material, especially since the last couple of decades have seen stricter controls over point sources of fecal pollution. It is generally argued that non-point sources including septic systems, agricultural runoff and wildlife among others are to blame due to the widespread nature of pollution and stringent controls over point sources. In order to effectively address these pollution incidents, current practice requires that individual sources of fecal pollution be identified within a watershed context. This management approach has proved challenging due to the presence of multiple non-point sources of fecal pollution at the watershed level which makes it difficult to isolate the influence of individual sources (Carroll et al., 2005). It is no surprise that most watershed management programs have failed to account for OWTS as a potentially prominent source of fecal pollution. This raises concern considering that several researchers have implicated OWTS in fecal pollution of surface, ground and marine water resources (Bremer and Harter, 2012; Cahoon et al., 2006; Habteselassie et al., 2011; Lipp et al., 2001). The need to isolate and quantify the impact of OWTS to inform watershed management efforts is therefore overdue. The overall goal of this study was to understand the dynamics of OWTS impacts at the watershed level as influenced by watershed level characteristics. To achieve this we assessed the temporal and seasonal influence of OWTS on the fecal bacteria levels of streams in watersheds impacted by high or low density of OWTS.

MATERIALS AND METHODS

Study Area

The study area has been described in detail in Landers and Ankcorn (2008). The area is in the Southern Piedmont region, southeast of Atlanta, GA (Figure 1) and has a mean annual precipitation of about 1278 mm. A summary of watershed characteristics are presented in Table 1. The selected watersheds are in the Ocmulgee and Oconee River basins, which drain to the Altamaha River and the Atlantic Ocean. These watersheds are in the Southern Appalachians region which is a sensitive ecological zone with 65% of rivers and streams in poor condition (US EPA, 2013). Out of the 24 selected watersheds, 12 are characterized as having high density of OWTS (HD) with the remaining 12 characterized as having low density of OWTS (LD). A watershed with less than 38 OWTS units per km^2 is considered as a LD while a watershed with greater than 77 OWTS units per km^2 was considered as HD (Table 1). Other watershed selection criteria included similar geological setting, precipitation, climate, accurate baseflow measurement locations and available spatial datasets of natural, infrastructure, and water-use characteristics. Watershed boundaries and monitoring locations are presented in Figure 1.

Sampling Regimes and Measured Parameters

Surface water samples from streams in the 24 watersheds were collected during baseflow conditions on 3 separate synoptic sampling events in 2012. Samples (n = 72) were collected in duplicate in 1 L bottles for each site in March, July and November to coincide with the 3 major seasons. Streamflow (velocity-area method) as well as standard water quality parameters (in-situ probe) such as pH, temperature, dissolved oxygen and specific conductance were measured during synoptic sampling events. Water samples were analyzed immediately for fecal indicator bacteria (*E. coli* and enterococci) using the Colilert and Enterolert kits (IDEXX Laboratories Inc., Westbrook, ME). Water samples were also filtered to collect DNA for later microbial source tracking work using quantitative polymerase chain reaction techniques to identify bacterial and viral genetic markers specific to different fecal pollution sources. The source tracking data are not presented here. All monitoring data was reviewed for normality and if necessary log-transformed to achieve normality prior to data analysis. All data was analyzed with the SigmaPlot (Systat Software Inc., San Jose, CA) statistical package and *p*-values estimated at the 95% confidence level.

RESULTS AND DISCUSSION

Standard water quality parameters including pH, dissolved oxygen and temperature were comparable between streams located in watersheds impacted by high or low density of OWTS (Table 2). Specific conductance on the other hand was consistently higher in samples from high density watersheds (Table 2). *E. coli* and enterococci concentrations varied by season in response to streamflow and the density of OWTS (Figure 2 and Table 2). Samples from high density watersheds showed consistently lower fecal bacterial concentration with decreasing discharge rate from March to November. No seasonal trend was observed in bacterial concentration for water samples from low density septic impacted watersheds (Figure 2).

Fecal bacterial yield, which accounts for the influence of streamflow and watershed area on bacterial concentration, was overall higher in HD than LD watersheds. Significantly, the dry summer season showed the greatest difference in bacterial yield between HD and LD watersheds (Figure 2). This period coincided with a 65% increase in baseflow yield in HD watersheds over LD watersheds. Statistically, *E. coli* and enterococci yields in HD watersheds were significantly different from the yields in LD watersheds for the summer (Table 3). Critical values (*p*-values) were 0.02 and 0.008 for *E. coli* and enterococci respectively. This result suggests the significant influence of OWTS density on microbial water quality under dry conditions. These results are consistent with observations by Landers and Ankcorn (2008) that, under dry conditions, septic systems can potentially impact streamflow and water quality. Furthermore, correlation analysis (Table 4) confirms an overall positive correlation between HD watersheds and bacterial yield. *E. coli* was strongly correlated with HD OWTS in the spring and summer seasons whilst enterococcus showed strong correlation with OWTS density during the summer and winter seasons. Moreover, specific conductance (a useful indicator of wastewater presence in water bodies) was positively and strongly correlated with high density of OWTS for all seasons. The data suggests that the relationship between septic density and water quality parameters is strongest with density above 88 units/km^2 (229 units/mi^2).

CONCLUSIONS

This study provides evidence of septic system impacts on water quality through the comparative analysis of water quality parameters in watersheds of varying septic system density. Correlation analysis indicates a potential association between septic system density and increasing bacterial yield at the watershed level. Importantly, available data points to strong septic system influence in watersheds with septic density exceeding 88 units/km^2. Even though the presented data confirms a positive association between septic systems and fecal pollution, other lines of evidence such as microbial source tracking need to be explored in order to improve confidence in the underlying conclusions. Finally, the data from this study can be used to evaluate the effectiveness of existing watershed management programs as well as inform the development of new target-oriented watershed management efforts.

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Tables and Figures

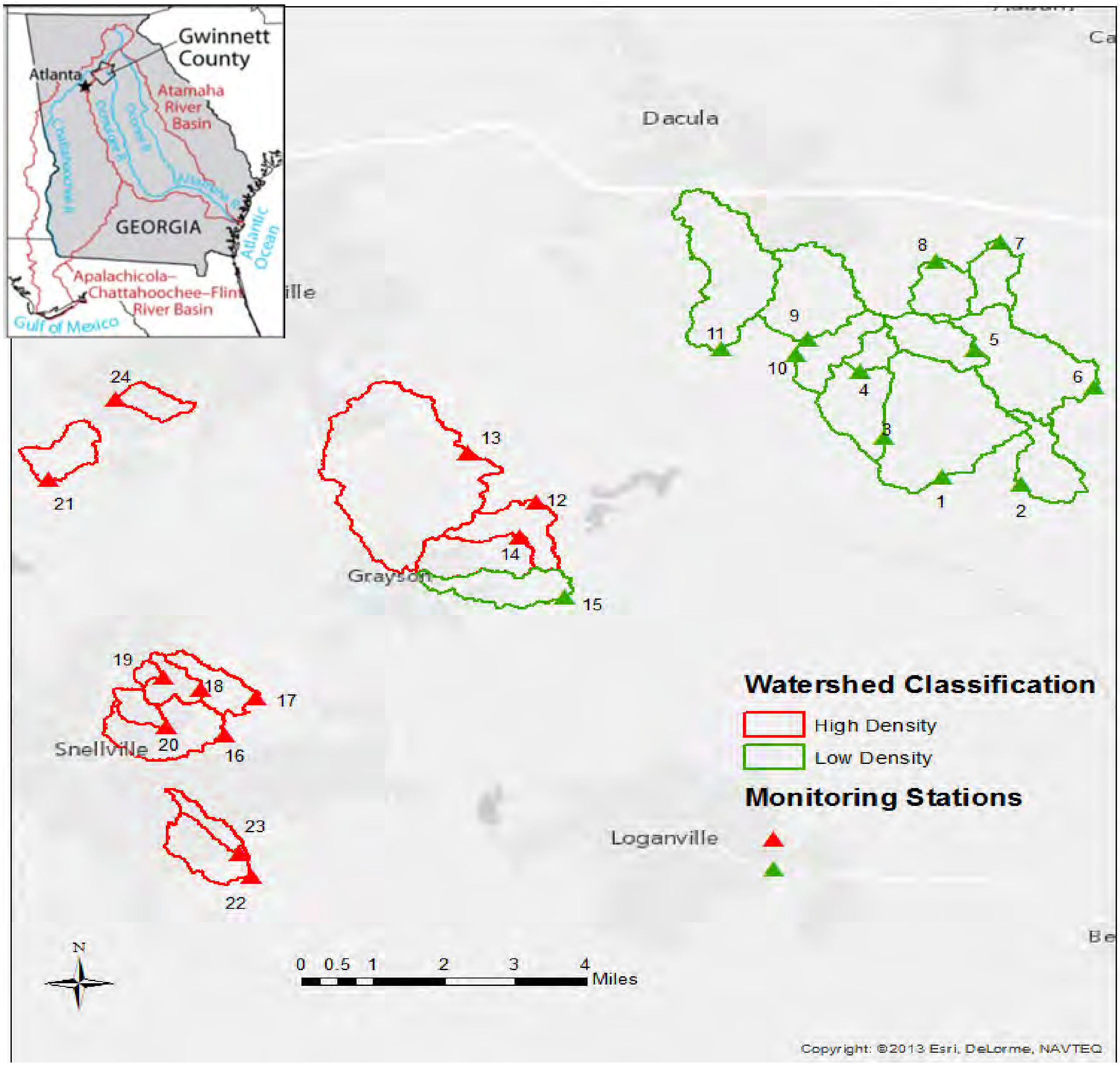

Figure 1 – Location of the study site with 24 watershed boundaries and monitoring stations in Gwinnett County, GA

Table 1 – Basic characteristics of watersheds in the study area

Watershed ID	Imperviousness %	OWTS Density (#/km^2)	Basin Area (km^2)	Slope %	Median distance OWTS to stream (m)
1	4.20	8	8.39	8.8	163
2	3.30	10	1.55	10.6	126
3	4.30	14	2.67	8.5	163
4	11.60	36	0.62	7.3	172
5	5.40	20	1.48	5.8	86
6	4.10	15	5.28	6.5	108
7	6.30	18	1.11	10.6	90
8	3.00	17	1.27	9.2	94
9	7.80	27	2.95	7.7	159
10	7.30	34	4.40	8.3	119
11	7.60	25	4.20	7.8	119
15	15.20	37	1.68	4.6	140
12	12.30	115	3.29	9.1	105
13	13.20	88	8.81	8	117
14	16.10	141	1.74	8.5	104
16	26.40	187	2.59	5.7	99
17	20.10	230	1.68	7.5	138
18	18.40	308	0.98	7.4	151
19	20.30	373	0.18	7.8	105
20	18.30	290	0.54	6	83
21	17.50	214	1.14	8.6	63
22	18.90	157	1.94	7	63
23	18.40	233	0.52	7.3	65
24	20.00	253	0.67	7.6	55
Mean LD†	6.68	22	2.97	8	128
Mean HD‡	18.33	216	2.01	7.5	96

†**LD: Low Density watershed** ‡**HD: High Density watershed**

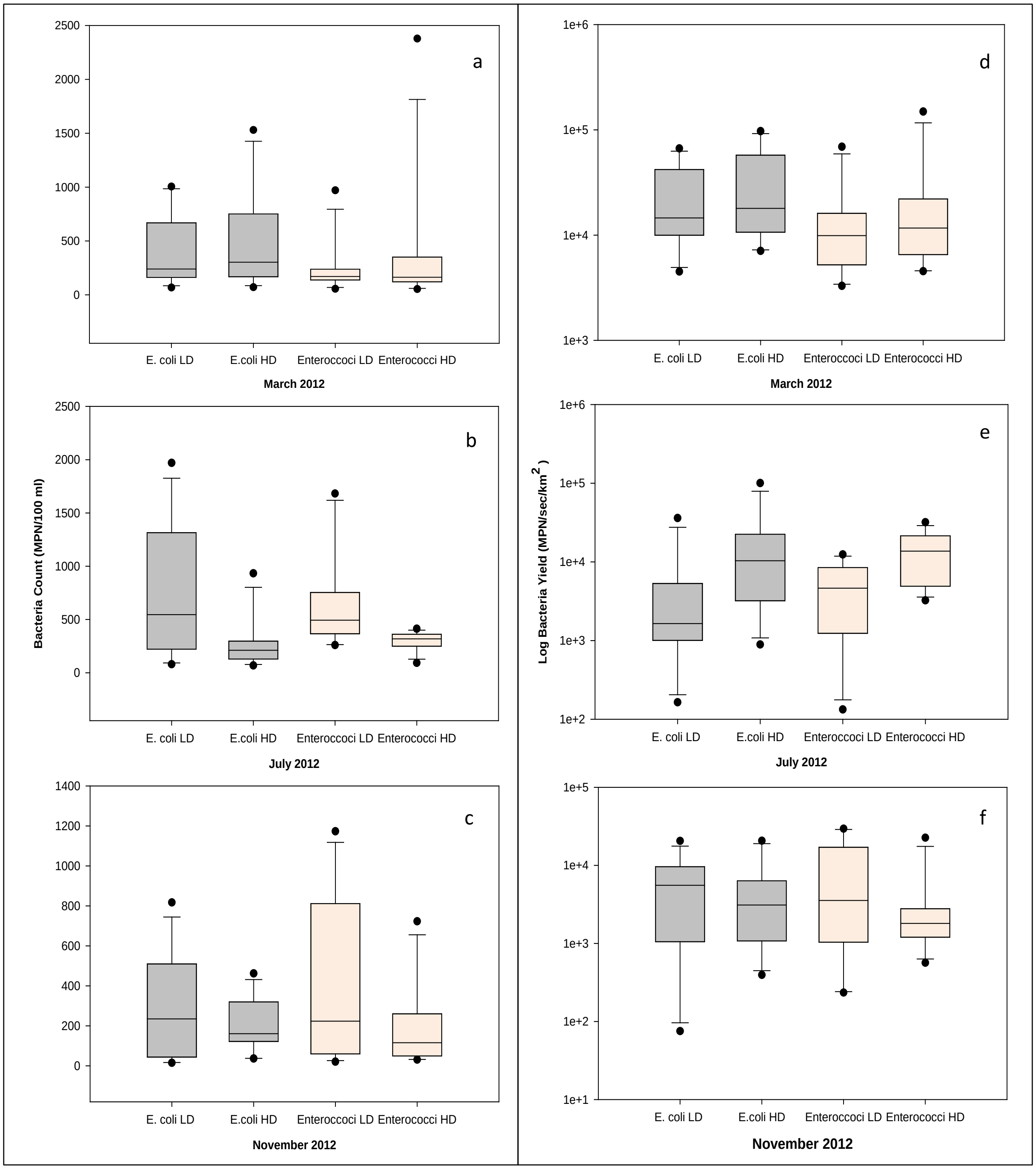

Figure 2 – Seasonal changes in bacteria concentration (a, b, c) and yield (d, e, f) in streams of high or low density of OWTS

Table 2 – Standard water quality parameters and discharge rates for high and low density watersheds								
Parameter	**Mar-12**		**Jul-12**		**Nov-12**		**Pooled Data**	
	Mean LD	**Mean HD**	**Mean LD**	**Mean HD**	**Mean LD**	**Mean HD**	**Mean LD**	**Mean HD**
pH	6.37	6.09	6.82	6.60	6.70	6.68	6.63	6.46
Temperature (^oC)	15.97	18.45	21.55	22.45	10.28	11.16	15.94	17.35
Dissolved Oxygen (mg/l)	8.05	9.33	7.63	6.85	8.49	8.77	8.05	8.32
Specific conductance (uS/cm)	57.94	121.11	46.72	81.43	45.75	70.17	50.13	90.90
Discharge (m^3/sec)	0.0207	0.0142	0.0029	0.00994	0.0065	0.0053	0.01	0.0098
Baseflow yield ($m^3/sec/km^2$)	0.0062	0.0069	0.001	0.0061	0.0021	0.0026	0.0036	0.0046

Table 3 - One-way ANOVA results (*p*-values) for water quality differences by OWTS density												
Parameter	**Mar-12**			**Jul-12**			**Nov-12**			**Pooled dataset**		
	Mean LD	**Mean HD**	***p*-value**	**Mean LD**	**Mean HD**	***p*-value**	**Mean LD**	**Mean HD**	***p*-value**	**Mean LD**	**Mean HD**	***p*-value**
E. Coli Yield ($MPN/sec/km^2$)	23918	33969	0.43	5186	18680	0.02*	6350	5197	0.97	11818	19282	0.17
Enterococci Yield ($MPN/sec/km^2$)	16596	25284	0.55	5019	14272	0.008*	8856	3718	0.37	10157	14425	0.31
Specific conductance (uS/cm)	58	121	<0.001*	47	81	<0.001*	46	70	0.002	50	91	<0.001*

* **Statistically significant**

Table 4 – Pearson correlation coefficient for water quality parameters vs. OWTS density								
Parameter	**Mar-12**		**Jul-12**		**Nov-12**		**Pooled Data**	
	Low Density	**High Density**	**Low Density**	**High Density**	**Low Density**	**High Density**	**Low Density**	**High Density**
E. coli Yield ($MPN/sec/km^2$)	-0.429	0.39	-0.565	0.315	-0.687	0.0584	-0.491	0.192
Enterococci Yield ($MPN/sec/km^2$)	-0.202	0.084	-0.669	0.233	-0.354	0.212	-0.369	0.121
Specific conductance (uS/cm)	0.698	0.437	-0.27	0.752	-0.462	0.667	-0.123	0.508

Minimum lot size estimates for nitrogen assimilation in onsite wastewater treatment systems

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ABSTRACT

State regulatory agencies set standards for minimum lot size for homes with onsite wastewater treatment systems (OWTS) based on the expected nitrogen (N) load to groundwater. However, the data to support these standards are sparse. In a recent field study on a clay soil, we developed a two-dimensional model for N treatment. Our objective was to use this model to estimate the minimum lot sizes that would be required for all 12 soil textural classes. The model was run for each soil textural class for two years using weather data for April 2009 to April 2011.The minimum lot size was calculated using an equation in the Georgia OWTS Manual. Denitrification losses varied widely among soils, ranging from 1% in the sand class to 75% in the sandy clay class. This was due to the effect of water content on denitrification. Leaching losses to groundwater ranged from 27% in the sandy clay class to 97% in the sand class. We found that it was important to consider differences in recharge among soil textural classes in estimating the minimum lot size to protect groundwater. The lot sizes ranged from 0.27 to 1.12 ha and were largest for sandy soils, but some medium textured soils also had large lot requirements.

Drainfield trenches in onsite wastewater treatment systems (OWTS) are used to distribute septic tank effluent and allow it to infiltrate into the soil. An OWTS can experience hydraulic failure if the effluent loading rate exceeds the infiltration capacity of the soil. Radcliffe and West (2009) proposed dividing soil textural classes into four groups with the design hydraulic loading rate (HLR_D) ranging from 1 to 4 cm/d based on simulations using a two-dimensional HYDRUS model (Šimůnek et al., 2006). OWTS can experience water quality failure if N concentrations in effluent leaching to groundwater are sufficiently high to cause groundwater concentrations of nitrate (NO_3^-) to exceed drinking water standards (10 mg/L NO_3^--N). State regulatory agencies have developed minimum lot size recommendations for OWTS based on estimates of N leaching to groundwater. OWTS have been identified as a source of NO_3^- in aquifers (Hinkle et al., 2007; Welhan and Poulson, 2009). However, estimates of the amount of NO_3^- leaching to groundwater are variable. Gold et al. (1990) reported NO_3^- concentrations taken from lysimeters installed 1 m below several OWTS exceeded 10 mg/L. In contrast, groundwater NO_3^- concentrations reported by Cogger and Carlile (1984) ranged <0.5 to 4.6 mg/L.

Recently, we calibrated a two-dimensional HYDRUS model using experimental soil pressure head and vadose zone N and chloride (Cl^-) data from a conventional OWTS installed in a clay soil in the Piedmont region of Georgia (Bradshaw and Radcliffe, 2013). An N chain model with water-content dependent first-order transformation rates for nitrification and denitrification was developed. The overall predicted soil pressure heads and solute concentrations were similar to data collected from the field experiment over a two-year period. The model was described in Bradshaw et al. (2013).

In Georgia, county health departments have the authority to set minimum lot sizes for homes with OWTS to prevent NO_3^- contamination of groundwater. The Georgia OWTS manual (GDPH, 2012) uses an equation to estimate the NO_3^- concentration in groundwater recharge from a home lot that can be written as follows:

$$n_r = \frac{V_w}{V_w + V_r}(1-d)n_w \qquad [1]$$

where n_r is the NO_3^--N concentration in mg/L in the recharge water, n_w is the total N concentration in the OWTS effluent, V_w is the wastewater discharge rate in L/d,V_r is the background groundwater (effluent-free) recharge rate in L/d, and d is the fraction of OWTS N that is lost to denitrification. V_r is the product of the lot area (cm^2) and the groundwater recharge rate (cm/d). The manual assumes that each bedroom generates 568 L/d (150 gal/d), the wastewater total N concentration (n_w) is 60 mg/L, and denitrification results in a loss of 50% of the effluent total N. Annual rainfall in Georgia is approximately 127 cm and the manual assumes that one half of this total becomes recharge. With these assumptions, the manual recommends a minimum lot size of 0.41 ha (1 acre) for a 4-bedroom home because the estimated groundwater recharge NO_3^--N concentration using Eq. [1] is 7.4 mg/L and less than the drinking water standard of 10 mg/L.

Our objective was to use the model developed by Bradshaw et al. (2013) to estimate the minimum lot sizes that would be required for the twelve soil textural classes.

MATERIALS AND METHODS

A detailed description of the model is given in Radcliffe and Bradshaw (2013). The OWTS model was developed using HYDRUS version 2.01 (Šimůnek et al., 2011). It is a finite element model that uses a numerical solution to the Richards (1931) equation to simulate variably saturated water flow in soil. We used the hydraulic properties from the HYDRUS Rosetta database (Schaap et al., 2001). The OWTS models were run for 17,760 h (740 d) using the precipitation and temperature data from 1 April 2009 to 10 April 2011 in the field experiment by Bradshaw and Radcliffe (2013). Solute transport in HYDRUS is described by a numerical solution to the advection-dispersion equation (ADE). Soil temperature was simulated based on the heat flow equation using default soil heat transport parameters in HYDRUS for a clay, loam, or sand, depending on the soil textural class being modeled (Šimůnek et al., 2011).

The OWTS model space consisted of a trench and the surrounding soil with one axis vertical and the other horizontal. One half of the drainfield was used for the model space assuming the middle of the trench was an axis of symmetry. The model space was 125 cm in the horizontal dimension and 150 cm in the vertical direction. The trench bottom was 72 cm below the soil surface, the depth of the trench bottom in the field experiment of Bradshaw and Radcliffe (2013) and a typical installation depth for the Georgia Piedmont region. The soil surface formed the top of the model space. The trench was 45 cm in width (half of a full trench) and 30 cm in height. The dose rate was chosen so that the effluent dose, expressed as a volume of effluent per area of trench bottom, was 4, 3, 2, or 1 cm/d, depending on the soil group category (I, II, III, or IV, respectively) (Table 1).

We used a two-solute N chain model consisting of NH_4^+ and NO_3^- . We assumed all the N in the effluent from the septic tank was in the form of NH_4^+. The transformation of NH_4^+ to NO_3^- (nitrification) was modeled as a single step, first-order reaction. Denitrification was modeled as a first-order reaction loss of NO_3.The values for rate nitrification and denitrifcation rate constants were set at 0.045 and 0.01 1/h, respectively, except in the lower soil horizon which was assigned a value for μ of 0.001 1/h to reflect the limiting effect of lower carbon levels on denitrification deeper in the soil profile. All of these values were based on the calibrated model of Bradshaw et al. (2013). Water content dependency functions limited nitrification and denitrification.

We used Eq. [1] to calculate the minimum lot size for a 4-bedroom home for each soil textural class that would result in a recharge concentration (n_r) of 10 mg/L. We assumed the same total N concentration for wastewater (60 mg/L) and discharge rate per bedroom (568 L/d) as the Georgia OWTS manual, but used the denitrification loss percentages that we found in the simulations (Fig. 1). We used two estimates of the groundwater recharge rate: 1) 50% of annual rainfall as in the OWTS manual and 2) the percentage of rainfall found in the recharge simulations for each soil textural class where the models were run without any input of OWTS wastewater. We used the average annual rainfall from the experiment by Bradshaw and Radcliffe (2013), 122 cm.

RESULTS AND DISCUSSION

The N mass balance for each soil class based on the two-year simulation of a mature OWTS is shown in Fig. 1. These results were presented previously in Radcliffe and Bradshaw (2013). The soil textural classes are listed from left-to-right in the order of of increasing denitrification loss percentage. The mass balance was good in that the residual was less 2% for all classes. There was a wide range in leaching losses (27-97%) and denitrification losses (1-75%), but plant uptake (0-4%) and change in storage (0-2%) were small and in a narrow range. Leaching losses decreased and denitrification losses increased progressing from Group-I to Group-IV soils. This was due to decreasing water contents below the trench as the dose rate decreased from Group-I to Group-IV soils (Table 1). As water contents decreased, denitrification was inhibited.

In Table 2, the soil textural classes are listed in the order of increasing denitrification loss percentage. Using 50% of rainfall as an estimate of recharge, the lot sizes ranged from 0.07 to 0.65 ha. Lot size decreased steadily from Group I to Group IV soils as the simulated denitrification percentage loss increased. The recommended minimum lot size of 0.41 ha (1 acre) in the Georgia OWTS manual was a conservative estimate for all soil classes except the sand and loamy sand.

Using the second method where differences in recharge rates among soil textural classes were considered resulted in higher values for the minimum lot size (ranging from 0.27 to 1.12 ha) and the pattern among soil groups was more complicated. Recharge percentages were highly variable and ranged from 13 to 44%, all less than the assumed rate of 50% rainfall in the Georgia OWTS manual. As expected, the highest recharge percentages occurred in the Group-I soils with high saturated hydraulic conductivity. The high recharge rates in this group offset the low denitrification rates in some cases. This can be seen in the Group-I silt which had the next-to-lowest denitrification rate, but only required a lot size of 0.54 ha due to the large recharge rate (42% of rainfall). The largest lot sizes occurred in sandy soils where denitrification rates were extremely low (Group-I sand and loamy sand). In contrast to the first method, lot size did not decrease as much for clayey soils because recharge was low in these soils due to low saturated hydraulic conductivities. Using the second method of calculating the minimum lot size, the Georgia OWTS manual recommendation is too low for all soils except the sandy clay and clay classes. This analysis shows the importance of accounting for differences among soil textural classes in recharge as well as denitrification.

CONCLUSIONS

Our simulations showed N treatment varied widely among the soil textural classes with denitrification losses that ranged from 1 to 75% and leaching losses that ranged from 27 to 97% of the total N input. States generally assume that denitrifcation losses are 25% or less (certainly less than 50%) so our results indicate a wider range among soils. The HLR_D grouping was a good predictor of N treatment in that the sandy Group I soils had the lowest denitrification (and highest leaching) losses and the Group IV clayey soils had the highest denitrification (and lowest leaching) losses. The primary reason for the denitrification differences was the difference in hydraulic performance and its effect on denitrification. Plant uptake and sorption accounted for 5% or less of the N input, perhaps due to the relatively deep installation depth. Minimum lot sizes designed to prevent groundwater concentrations of NO_3^--N above 10 mg/L varied widely among the soil textural classes, ranging from 0.27 to 1.12 ha, and were higher for most soil classes than the minimum lot size recommended in Georgia (0.41 ha). Our simulations showed that it was important to consider the effect of soil texture on recharge as well as denitrification and that some medium textured soils had large lot size requirement.

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Table 1. Hydraulic group, design hydraulic load (HLR_D) , Group dose rate, and saturated hydraulic conductivity (K_s) for twelve soil textural classes.

Textural class	Group†	HLR_D† cm/d	Dose Rate cm/d	K_s cm/d
Silt	I	5.40	4	43.74
Sand	I	5.16	4	642.98
Silt loam	I	4.71	4	18.26
Loamy sand	I	4.44	4	105.12
Sandy loam	II	3.31	3	38.25
Silty clay loam	II	2.97	3	11.11
Loam	II	2.79	3	12.04
Sandy clay loam	III	2.08	2	13.19
Clay	III	2.02	2	14.75
Clay loam	III	2.00	2	8.18
Silty clay	III	1.91	2	9.61
Sandy clay	IV	1.48	1	11.35

†From Radcliffe and West (2009).

Table 2. Minimum lot size for 12 soil textural classes. Minimum lot size was calculated in two ways: 1) assuming that recharge was 50% of rainfall and 2) using recharge percentage of rainfall from model simulations. Model recharge percentage is also shown.

Textural class	Group	Minimum Lot Size 50% Rainfall ha	Minimum Lot Size Model Rainfall ha	Model Recharge %
Sand	I	0.65	0.74	44%
Loamy sand	I	0.58	1.12	26%
Silt	I	0.45	0.54	42%
Silt loam	I	0.37	0.68	27%
Sandy loam	II	0.35	0.91	19%
Silty clay loam	II	0.26	0.64	20%
Loam	II	0.25	0.63	20%
Clay loam	III	0.20	0.71	14%
Sandy clay loam	III	0.20	0.59	17%
Clay	III	0.16	0.48	16%
Silty clay	III	0.16	0.60	13%
Sandy clay	IV	0.07	0.27	13%

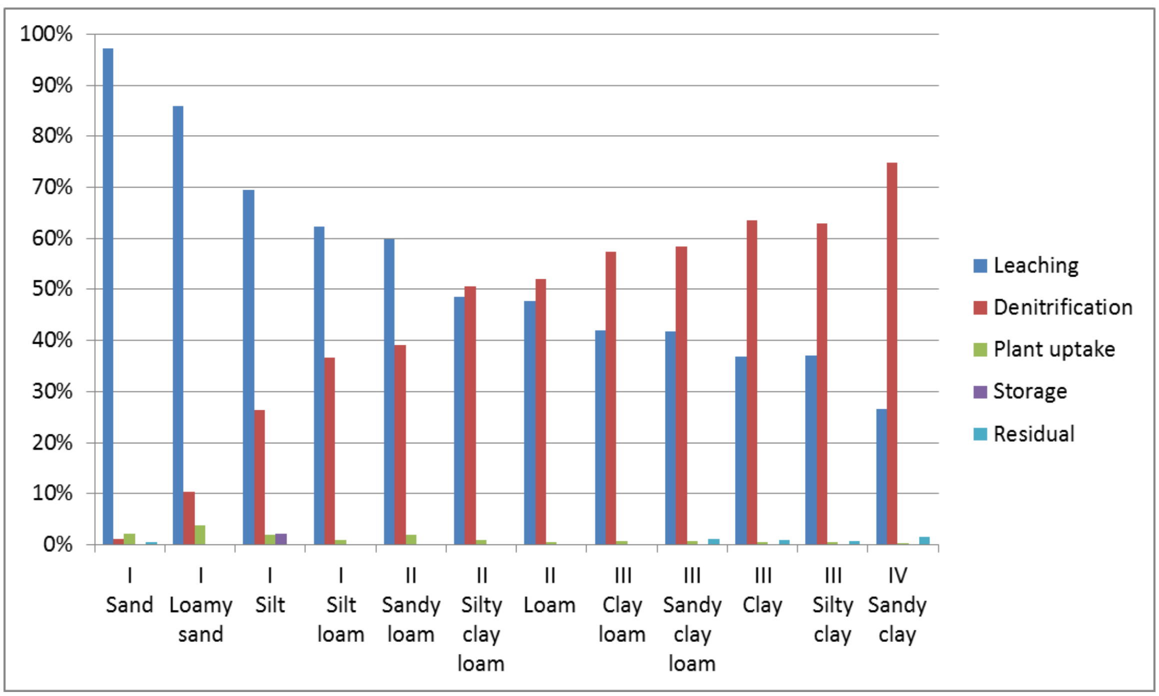

Figure 1. Nitrogen mass balance for the various soil textural classes in the two-year simulation for a mature OWTS.

Performance of Riparian Buffers around Onsite Systems in Suburban Settings

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ABSTRACT

Nonpoint source (NPS) pollution has been recognized as the leading cause of water quality problems in the United States. Onsite wastewater management systems are among the nonpoint sources that may results in water quality degradation. In a properly functioning septic system, wastewater infiltrating the soil receives treatment in the unsaturated zone before reaching the saturated zone (i.e., groundwater) or reaching a slowly permeable layer where a saturated zone or perched water table may form. In this study, we assessed the movement of water and the selected constituents of domestic wastewater through the soil in the buffer areas between three septic system drainfields and natural streams. Two neighboring septic systems serving single family homes had drainfields located near a small creek. The other individual septic system was located at a distance of 150 m from a major creek. The buffer areas for the systems were in flood plains adjacent to the creeks. The soil at each site was characterized by hand auger boring, and a series of wells, piezometers, tensiometers and time domain reflectometry (TDR) rods were installed at different locations inside and outside the drainfield of each system. Soil water content and pressure head of the unsaturated zone at different depths, and depth to water table and submergence potential at two depths were measured biweekly. Water samples from the groundwater were collected once a month using wells and piezometers. In addition, soil solution samples were collected using tension lysimeters, and water samples from different locations along the neighboring creeks were collected for analysis. On average, the amount of wastewater applied to the drainfield of each of the systems was less than 40% of the daily design flow for that system. Although soil water content under the drainfield of each system was relatively high during part of the year, the soil remained unsaturated allowing the systems to hydraulically function properly. With few exceptions, the concentrations of both nitrate-nitrogen (NO_3-N) and ammonium-nitrogen (NH_4-N) in the creek water were less than 0.5 mg/L. Ammonium-N concentrations as high as 5 mg/L were occasionally measured in samples of well water and soil solution collected from the drainfield areas of the systems. Nitrate-N concentrations greater than 5 mg/L, however, were observed frequently in groundwater and soil solution samples. Based on these results, it appears that denitrification and dilution within the saturated zone were the primary mechanisms for low concentration of nitrogen compounds in the creek water at these sites.

Keywords: Nitrate, ammonium, phosphate

Nonpoint source (NPS) pollution has been identified as the nation's largest cause of water quality problems (USEPA, 2014), and septic systems are considered to be one of the major sources (Bicki and Brown, 1991; Hantzsche and Finnemore, 1992; Tinker, 1991; Wernick et al., 1998). Approximately 20% of the households in the United States and almost one-half the population of North Carolina use on-site systems for wastewater management (US Census Bureau, 2011, 2004). Based on the estimated North Carolina population of 9,848,000 (Bureau of Census, 2014), and assuming an average daily water use of 260 L per individual (USEPA, 2002), the volume of wastewater applied daily to North Carolina soils through septic systems exceeds 1.2 billion L (3.2×10^8 gallons).

While a great deal of attention has been devoted to nonpoint sources resulting from agricultural operations (Domagalski and Johnson, 2011; Eghball and Gilley, 1999; Mostaghimi et al., 1992; Peterson and Benning, 2013; Steinheimer et al., 1998), only a limited number of studies have addressed the impact of septic systems on the quality of surface and groundwater at watershed scales (Wernick et al., 1998; Katz et al., 1980; Tinker, 1991). Reneau et al. (1989) presented a review of the fate and transport of contaminants in soils under septic systems and concluded that transport of nitrate and perhaps ammonium pose the greatest pollution potential from septic systems. Robertson et al. (1991) found significant movement of a nitrate plum under a 14-year old and a < 3-year old septic systems serving single-family homes. Chen and Harkin (1998), however, reported that nitrate-N from different sources contributed to groundwater and concluded that septic systems are not a dominant source of nitrate in groundwater down gradient from septic systems. In North Carolina, Morey and Amoozegar (2004) reported relatively high levels of nitrate (up to 18 mg/L) in water samples from the top of the shallow groundwater under the (mound) drainfield area of a small septic system installed in a sandy soil. Cogger et al. (1988) showed that one-ft (30 cm) of separation distance (unsaturated soil) between the bottom of the trenches and water table in sandy soils of the Lower Coastal Plain Region is not adequate for treating septic tank effluent. Stall et al. (2014) confirmed that 60-cm of unsaturated flow is most effective in removing *E. coli* but neither 30 nor 45 cm of unsaturated flow is adequate for treating microbes in a loamy sand soil.

The amount of N applied to soils through septic systems in large residential areas (e.g., subdivisions) may be considered excessive. For example, based on the estimated nitrogen concentration of 26 to 75 mg/L in septic tank effluent, the amount of N that can potentially be applied to the drainfield for a modest application rate of 0.125 gal/(ft^2d) [equivalent to 0.5 cm/d or 5 L/(m^2d)] is equivalent to 475 to 1,370 kg/ha (422 to 1,220 lbs/acre, respectively), which is much higher than fertilizer application rates used in agricultural operations.

Population increase, coupled with economic benefits of using septic systems in unsewered areas, will increase the use of on-site wastewater management systems. At the same time, the increased awareness regarding water quality, particularly in sensitive watersheds such as the Neuse River Basin in NC, demands more knowledge about the efficacy of soils to treat septic tank effluent. The overall goal of this study was to assess the movement of water and selected constituents of domestic wastewater through soils in the buffer areas between the drainfield of a number of septic systems and natural streams.

MATERIALS AND METHODS

Three individual septic systems within the Neuse River Basin in northern Wake County, NC, were selected for the study. System 1, served a 3-bedroom home with a design-loading rate of 1,360 L/d (360 gal/d). Systems 2 and 3 served 4-bedroom homes with a design-loading rate of 1,815 L/d (480 gal/d). Systems 1 and 2 were near each other on neighboring properties and were relatively close to a running creek. System 3 was installed at a distance of more than 150 m from a major creek that was located on the border of the property. Septic tank effluent from System 1 was dispersed within two subdrainfields by a low-pressure pipe (LPP) distribution system. For System 2 wastewater was applied to the drainfield through a pressure manifold and for System 3 wastewater entered the drainfield by gravity. All three systems were installed in grassy areas surrounded by mature trees.

The sampling scheme used for System 1 is shown in Figure 1 and a similar design was used at the other sites. The drainfield of System 1 was located in a relatively low area in front of a small creek on the property. The soil in the drainfield area of each system was described in the field and a series of soil samples were collected from the soil surface to a depth of 200 cm or deeper for particle size analysis. Saturated hydraulic conductivity (K_{sat}) of three depth intervals in the unsaturated zone within the drainfield areas of Systems 1 and 2 was measured in situ by the constant head well permeameter method (Amoozegar and Wilson, 1999). The infiltration rate within the two drainfield areas was also measured by the double-cylinder infiltrometer technique.

A series of time domain reflectometry (TDR) rods, and banks of tensiometers were installed at various depths and locations inside and outside the drainfield area of each system for measuring soil water content and pressure head (i.e., matric potential), respectively. To collect soil solution samples, tension lysimeters (here after referred to as lysimeters) were installed at two depths in two locations near a trench in each drainfield. Three observation/sampling wells for determining water table and collecting groundwater samples, and three to six banks of piezometers (two at different depths) for measuring submergence potential and collecting groundwater samples from different depths were also installed at each site.

All three sites were visited biweekly for recharging the tensiometers if needed. One day after this visit, each site was revisited and the water meter for the house was read and soil water content at different depth intervals, soil water pressure head at various depths and locations in the unsaturated zone, and the level of water in each piezometer and observation well were measured. During the first visit of each month, the water in each well and piezometer was bailed out and fresh groundwater was allowed to flow in for sampling. Using a hand vacuum pump, tension was applied to each lysimeter for collecting soil solution from the unsaturated zone. During the next day visit, a water sample was collected from each well and piezometer, and the content of each lysimeter was collected. In addition, a water sample was collected from each of the sampling locations along the creek at each site. The samples were transported in an ice box to the laboratory and analyzed for pH, electrical conductivity (EC), NH_4-N, NO_3-N, total Kjeldahl nitrogen (TKN), PO_4-P, total P (TP), and total organic carbon (TOC). Because of space limitation in this paper, only selected results will be discussed in detail. For additional information for all three systems see Amoozegar et al. (2004).

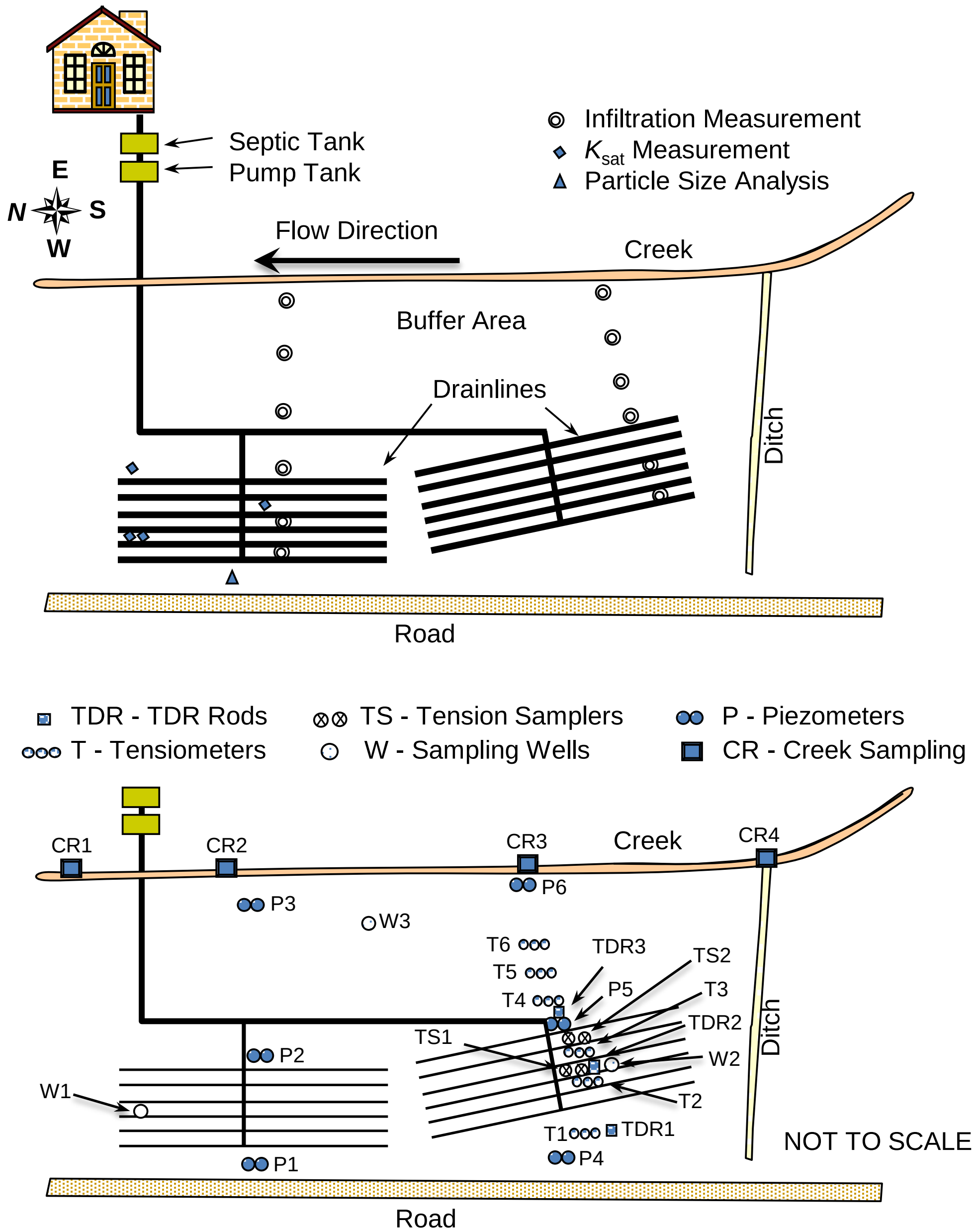

Figure 1. Schematic diagram of the plan view of the drainfield area of System 1 showing relative locations of the drainlines, particle size analysis samples, saturated hydraulic conductivity (K_{sat}) and infiltration rates measurements, and various monitoring devices.

RESULTS AND DISCUSSION

The soil in the drainfield area of System 1 appeared to be uniform. With a few exceptions, the soil texture was sandy loam from the surface to 150 cm depth and sand to loamy sand between 150 and 200 cm depths. Although the soil texture did not change significantly with depth in the upper 150 cm, the K_{sat} values were in general higher in the upper 40 to 60 cm than deeper depths. The lowest measured K_{sat} (1.6 cm/d) was for the 90 to 105 cm depth interval and was four times the area loading rate for the system. The infiltration rate in the upper part of the drainfield was lower than areas near the creek, but because of relatively high infiltration rate, we expected most of the rainfall to infiltrate and move vertically into the shallow groundwater at this site.

For System 2, the soil in the drainfield area also appeared to be uniform. The soil texture was sandy loam in the upper 40 to 45 cm depths, loamy sand between 45 and 80 cm, and sand to loamy sand with very little clay below 80 cm depth. The lowest measured K_{sat} was 3.5 cm/d at the 60 cm depth. The saprolite at approximately 105 to 120 cm depth interval had much higher hydraulic conductivity. In general, K_{sat} of soil at this site was high enough not to cause hydraulic failure. The infiltration rate for the area between the drainfield and the creek was relatively high and ranged between 0.9 and 9.7 cm/h. Except for very high intensity rainfall, we did not expect any potential runoff from the drainfield to flow directly into the creek.

The soil in the drainfield area of System 3 was relatively thick, and the clay content increased with depth, reaching a maximum of approximately 35% at the 90 to 110 cm depth interval. In the C (saprolite) horizon, the clay content decreased to an average value of 5% at 210 cm depth.

According to the water meters, the average values for the volumes of wastewater applied daily to the drainfields were 530 L for System 1, 640 L for System 2, and 608 L for System 3. These volumes represented approximately 40, 35, and 33% of the design flow for Systems 1, 2, and 3, respectively. Overall, considering the relatively high K_{sat} of the soils and low volume of wastewater produced by each household, we believe all the applied wastewater infiltrated the trenches and moved vertically into the groundwater at each site.

In general, the water table elevations at all three sites fluctuated with the seasons. At System 1 (Fig. 2), the water table in the middle of one subfield (marked as W2) was mostly above the bottom of the creek. Near the creek (marked as W3) and at the edge of the other subfield the water table was below the bottom of the creek for a few months during dry periods. Higher water levels in the middle of the drainfield could be the result of mounding due to wastewater application. Based on a three-point water table analysis, groundwater flowed from the drainfield toward the creek during the wet periods, while at other times, groundwater flowed mostly parallel to the creek. Overall, there was a good agreement between the groundwater table elevation in the wells and pressure heads measured in the piezometers at two depths (data not shown). There was virtually no difference between the total hydraulic heads at two different depths at two locations near the subfields. The highest difference in the total heads in the vertical direction was observed above the drainfield. Based on our observation of ground wetness at the south-west corner of the drainfield area, it appears that this area was a groundwater discharge area from up slope of the drainfield. We do not believe significant amounts of pollutants from this drainfield can move toward the creek when the water table is low.

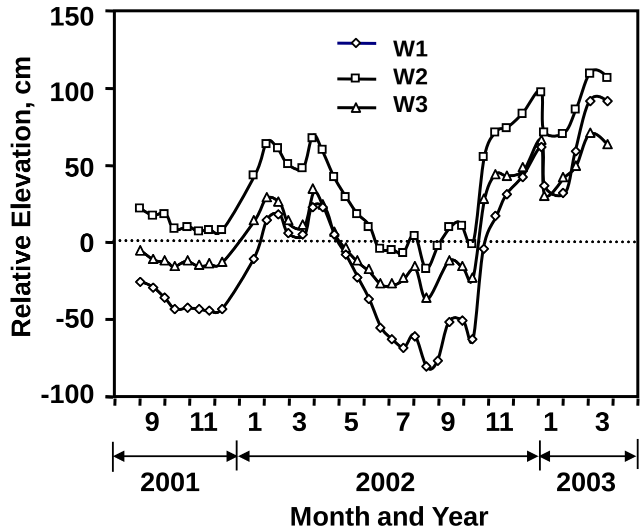

Figure 2. Water table relative elevations at three locations from September 2001 to April 2003 for Site 1. The elevation of the bottom of the creek at the location marked CR4 in Fig. 1 was selected as the reference location with zero elevation (shown as dashed-line).

For system 2, the water table elevation near the creek was generally lower than the water table at locations near the drainfield (data not shown). Only for a short time the water level elevation near the creek fell below the bottom of the creek. In general, the groundwater flow at this site was from the drainfield area toward the creek in a north-east direction (perpendicular to the general contour of the land). Overall, there was little difference between the water level elevations in the piezometers installed at different depths. The piezometer data indicated that the water from the drainfield area moved toward the creek most of the time.

All three observation/sampling wells at System 3 were below the drainfield. The differences in the water level in these wells (data not shown) indicated that groundwater moved from the north-west toward the south-east direction. It appears that groundwater from this system did not move directly toward the creek on the south side. The patterns for the water level elevation in the three piezometers were similar to the patterns for the water table elevation measured in the wells. Overall, the results for water levels in the piezometer tubes corresponded well with the water table elevations measured in the wells at the three locations.

The soil water content measured in situ at System 1 showed moderate variability with depth and time for each of the locations (Fig. 3). At the location above the drainfield, the wettest zone was the 90 to 120 cm depth interval, and the soil water content at each depth interval was relatively low during the summer months and relatively high during the winter when evapotranspiration is low. In the middle of the drainfield, water contents in the upper 45 cm showed more variation than water contents at the 60 to 90 cm depth interval. The trenches of this system were approximately 45 cm deep, and the water table was relatively shallow under the drainfield. As a result, the water content at the 60 to 90 cm depth interval was the highest and remained relatively constant. The average water content at 45 to 60 cm depth interval was 0.34 m^3/m^3.

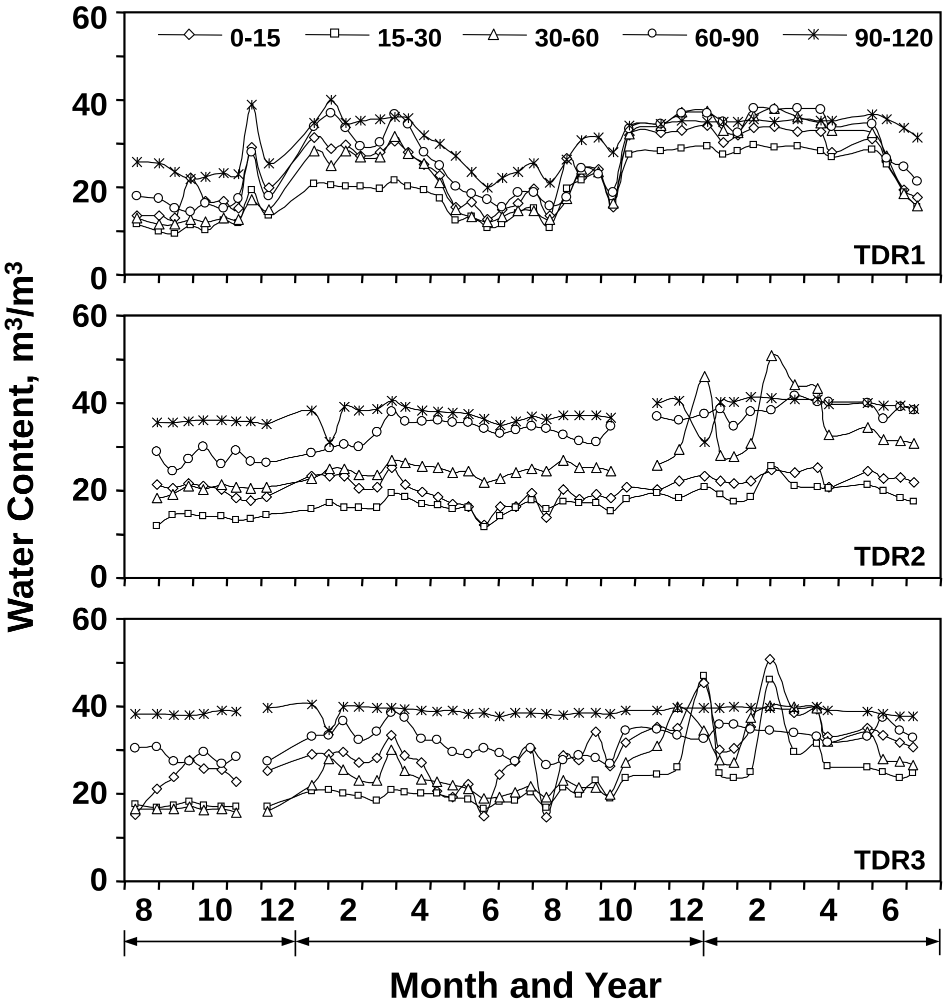

Figure 3. Soil water contents at five depth intervals measured in situ by the TDR technique at three locations at Site 1. For locations of the TDR measurements see Fig. 1.

At a location above the drainfield, the soil water pressure head in System 1 was close to zero or slightly negative from December 2001 through April 2002, and from November 2002 through early March 2003 (Fig. 4). At other times the soil water pressure head was mainly negative. Inside the drainfield, the soil water pressure head was mostly negative, indicating unsaturated conditions. Although there was not a substantial difference between the summer and winter months, the soil water pressure head was generally lower in the summer months as compared to winter months. Also, soil water pressure heads increased with depth, indicating higher soil water contents at deeper depths. At locations between the drainfield and the creek, the pattern of soil water pressure head distribution with time was relatively similar to the patterns for the two locations inside the drainfield. The trends of the soil water pressure head corresponded with the soil water content measured by TDR.

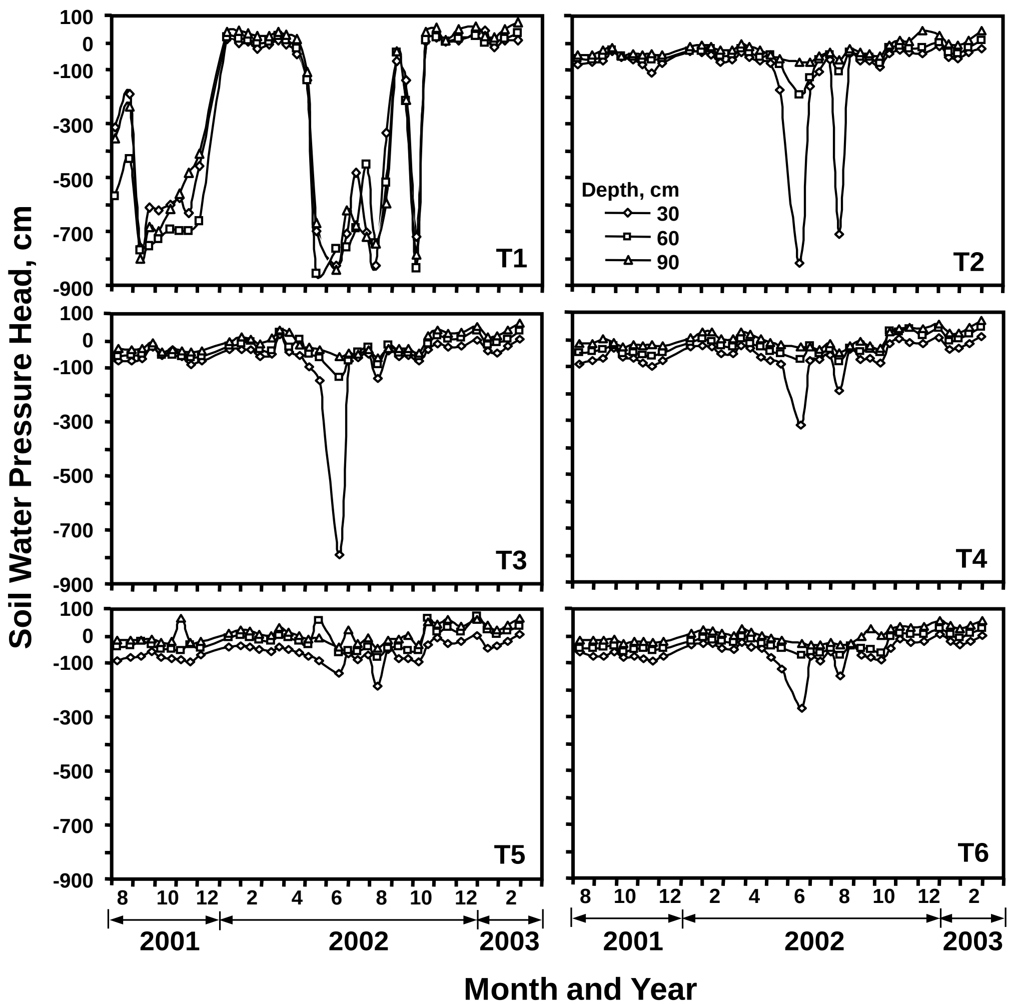

Figure 4. Soil water pressure heads for three depths at six locations along a transect going through one of the subfields at Site 1. The number in each graph represents the location of the tensiometer bank (see Fig. 1).

For System 2, the soil water pressure head measured at three depths above, in the middle, and below the drainfield near the beginning of the trenches corresponded fairly well with the corresponding soil water content measured by TDR. At locations above and below the drainfield higher soil water contents and lower pressure heads were measured during the winter months compared to summer (data not shown). Inside the drainfield, however, there was not a substantial difference in water contents between the summer and winter months, and the variation in soil water pressure head was much less from winter to summer. These results were expected because wastewater was applied to this drainfield throughout the year. Except for a few short periods, the soil in the drainfield area of this system remained unsaturated during the monitoring period. As indicated earlier, this system received approximately one-third of the daily design flow of 1815 L/d (480 gal/day). Also, the drainfield was located on a side slope and groundwater within the drainfield was relatively deep. Hydraulically, this system functioned properly by maintaining an unsaturated zone below its trenches.

The depth of ponding in System 3 was relatively low near the beginning of the trenches, and the highest level of ponding was observed at a location in the middle of the upper trench. Higher soil water content was also measured above the drainlines. These results were perhaps due to runoff that came from a large driveway sloping toward the drainfield area. Although ponding was observed continuously in part of each trench, the system appeared to function properly. Except for the upper 15 cm, the soil water content above the drainlines did not change substantially during 18 months of monitoring. Inside the drainfield, the water contents at the 60 to 120 cm depth interval remained relatively high at all times. The driest location was below the drainfield. Overall, the soil water pressure head results were consistent with the soil water content values and the level of wastewater ponding in the upper trench near these locations.

The pH of the water samples collected from the creek near the drainfields of Systems 1 and System 2 varied between 5.7 and 6.8. For both systems, the groundwater and soil solution samples had a lower pH than the surface water in the creek. In general, the pH values for the water samples collected from wells fluctuated more than the ones for the piezometers. For System 3, the pH of the water in the creek varied between 6.0 and 7.2 for the monitoring period. The results were consistent with the pH for the surface water in the creek at the other two sites a few miles away. The pH of the soil solution samples collected by the lysimeters varied between 6.0 and 7.3, while the groundwater pH was substantially lower and remained below 6.5.

Electrical conductivity represents the amount of total dissolved solutes in a solution. The EC of the water samples collected from seven locations in the creek adjacent to Systems 1 and 2 remained relatively low and did not vary substantially for the duration of monitoring. The EC of the water samples collected from wells and piezometers, as well as the soil solution, was higher than the EC of the creek samples. The results from these two systems indicate that dilution is perhaps the main reason for the reduction in the solute concentration in groundwater moving from the drainfield toward the creek. For System 3, the EC of water samples collected from the main creek on the south side of the property was slightly higher than the water samples from the side creek. For this system, the EC of the groundwater samples was relatively low, but occasionally was above 100 μS/cm. The EC the soil solution collected from the unsaturated zone near the trenches at two locations were more than 100 μS/cm for most samples.

Except for two sampling periods, the ammonium-N (NH_4-N) concentrations in the water samples from seven locations in the creek near Systems 1 (Fig. 6) and 2 remained relatively small. In the unsaturated zone of both systems, NH_4-N concentrations remained less than 1 mg/L for most of the monitoring period (Fig. 7). Low levels of NH_4 in the soil solution collected near the trenches indicated that the environment around the trenches remained aerobic during our study. In the groundwater, relatively high levels of NH_4 were only observed in the well that was installed in the middle of one of the subfields of System 1 (Fig. 8). Overall, the levels of NH_4 in the water samples collected from two different depths using the piezometers remained relatively low for both systems. For System 3, the NH_4-N concentrations in the water samples collected in the creeks were relatively low and with one exception did not exceed 0.5 mg/L. The NH_4-N concentrations in the well samples generally remained below 0.4 mg/L. A similar trend was observed in the samples collected from the piezometers. Ammonium-N concentrations in the soil solution samples collected by the lysimeters were less than 0.5 mg/L, but increased to more than 5 mg/L only once in two of the lysimeters.

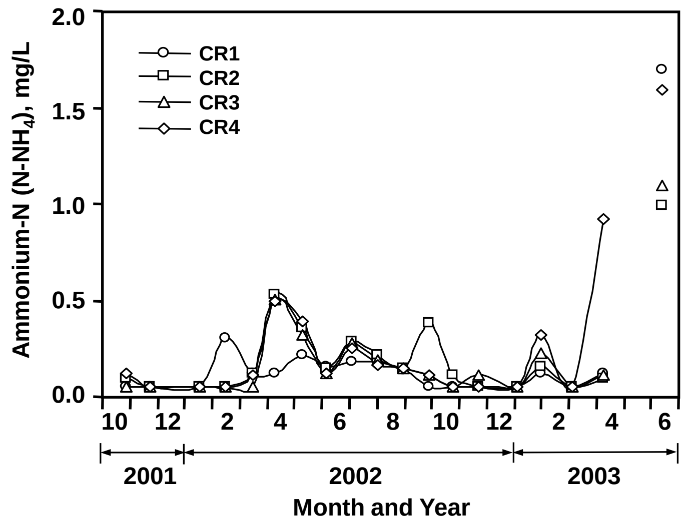

Figure 6. Ammonium-N (NH_4-N) concentrations in water at four locations in the creek adjacent to the drainfield of System 1. For sampling locations see Fig. 1.

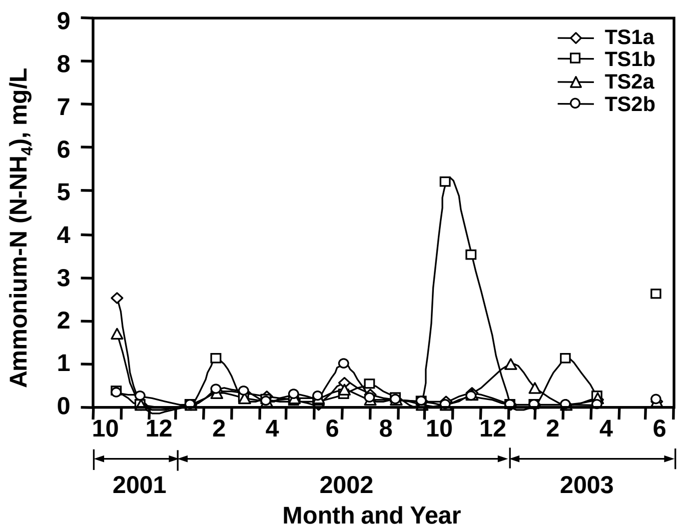

Figure 7. Ammonium-N (NH_4-N) concentrations in soil water collected from two depths at two locations near the trenches of System 1. In this figure "a" represents the lysimeter on the side of the trench and "b" represents the lysimeter at 20 cm below the bottom of the trench. For sampling locations see Fig. 1.

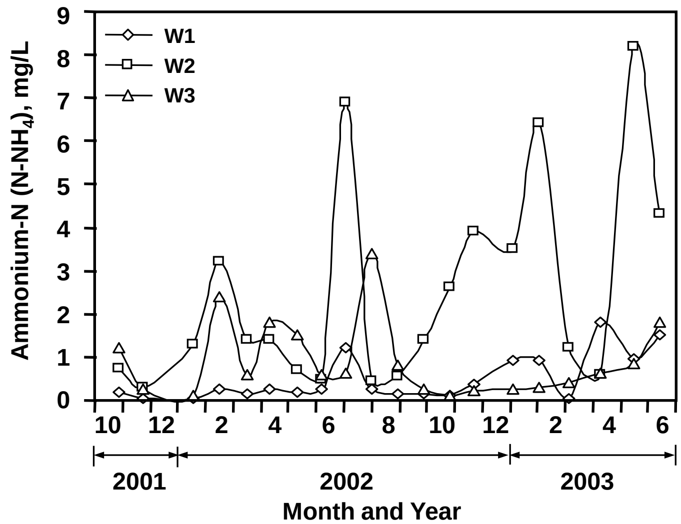

Figure 8. Ammonium-N (NH_4-N) concentrations in groundwater collected from three sampling wells in the drainfield area of System 1. For sampling locations see Fig. 1.

The NO_3-N levels in the creek at System 1 remained below 0.5 mg/L for the duration of monitoring (Fig. 9). The nitrate-N (NO_3-N) levels in the groundwater samples collected from the wells and the lysimeters, however, were substantially higher than their corresponding NH_4-N levels (Fig. 10). Higher concentrations of nitrate were observed during the winter and spring months than during summer and fall. This could be due to higher nutrient uptake by plants, or higher microorganism activities, although the possibility of higher leaching and a greater dilution due to summer rains cannot be ignored. Overall, lower concentration of nitrate in the creek compared to the soil and groundwater is perhaps due to denitrification and/or dilution within the buffer area between the system drainfield and the creek.

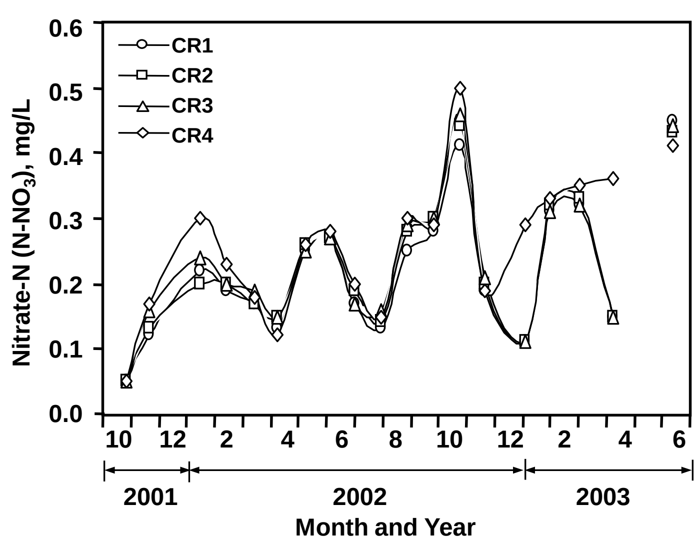

Figure 9. Nitrate-N (NO_3-N) concentrations in water samples collected at four locations in the creek adjacent to the drainfield of System 1. For sampling locations see Fig. 1.

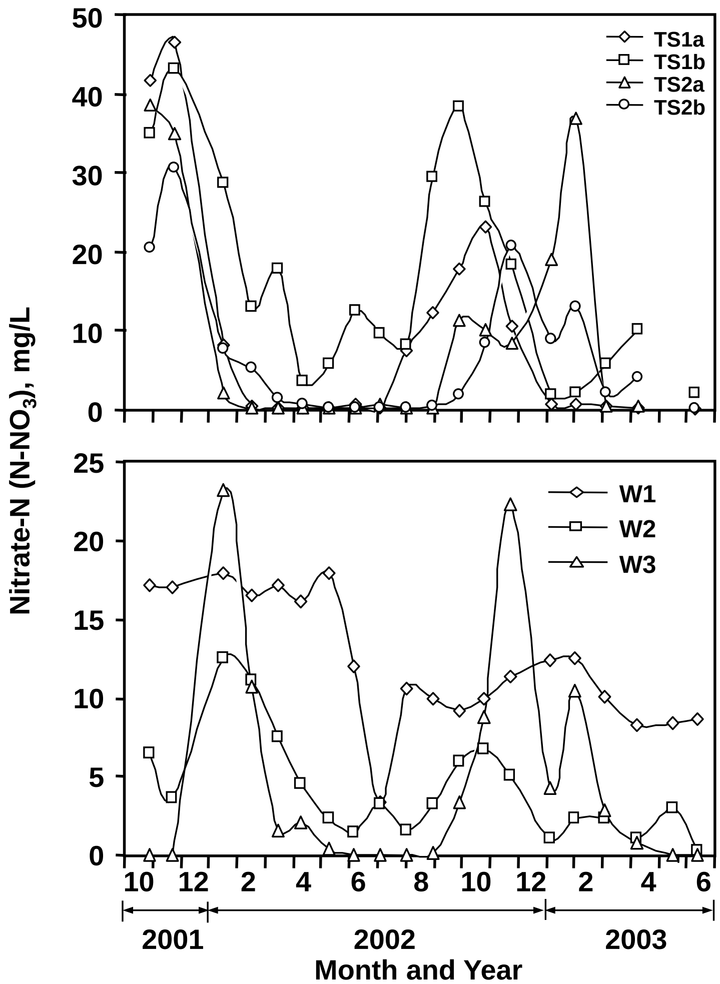

Figure 10. Nitrate-N (NO_3-N) concentrations in soil solution collected by lysimeters (TS) and in groundwater collected from three sampling wells (W) in the drainfield area of System 1. In this figure "a" represents the lysimeter on the side of the trench and "b" represents the lysimeter at 20 cm below the bottom of the trench. For sampling locations see Fig. 1.

For System 2, the levels of nitrate in the water in the creek were relatively low and showed the same trend as the locations along the creek adjacent to the drainfield of System 1. The levels of nitrate in the soil solution collected from near the trenches by the lysimeters, on the other hand, were substantially higher, but did not show any specific trend. The NO_3-N concentrations in the groundwater samples collected from the wells were generally low and reached 25 mg/L in two of the wells only once during the summer of 2002. Overall, the concentrations of nitrate in the soil solution and in groundwater under this system were lower than the corresponding values for System 1.

For System 3, the concentration of NO_3-N in the creeks was very low and did not exceed 0.4 mg/L during the monitoring period. In the soil solutions collected from the lysimeters, on the other hand, NO_3-N concentration reached as high as 18.5 mg/L. Overall, the soil solution had higher nitrate concentration than the groundwater. Nitrate-N concentrations in the samples collected from the wells and piezometers remained below 7 mg/L, but showed relative increases during late spring 2003. Overall, based on the ammonium and nitrate in the samples from near the trenches it appears that this septic system was functioning properly by maintaining an unsaturated zone below the trenches and converting the ammonium in the septic tank effluent to nitrate. Lower nitrate concentration in the groundwater compared to soil solution samples perhaps indicates reduction in nitrate concentration by dilution or denitrification.

The levels of phosphate-P (PO_4-P) in the groundwater and soil solution samples for System 1 rarely exceeded 1 mg/L. The total P concentrations in the water samples from the wells or lysimeters were also less than 1 mg/L for most of the times. Only for the piezometer samples did the total P levels exceed 1 mg/L a few times. For the samples collected from the creek near this system the concentrations of PO_4-P and total P were less than 0.1 and 0.16 mg/L, respectively, for the entire monitoring period. Similarly, the PO_4-P concentrations in the creek samples for System 2 were generally below the detectable limit and never reached 0.08 mg/L. With one exception, the PO_4 levels for the soil solution and groundwater samples were very low during the monitoring period. With one exception, the total P concentrations in the water samples from the creeks near System 3 were less than 0.1 mg/L. For this system, the total P concentration in the soil solution samples matched the phosphate concentration for individual sampling, indicating that most of the P in the soil solution was in the form of phosphate. For well and piezometer water samples, the total P was higher than phosphate-P.

The concentrations of total organic carbon (TOC) for all the samples collected from the creeks, lysimeters, wells, and piezometers at all three sites showed an interesting trend. In all cases, a substantially higher TOC concentration was detected from the samples collected in late 2001 and again from the samples collected from late September through December 2002. Because we have limited data, we cannot firmly substantiate a reason for higher TOC concentration. We could hypothesize that the results were faulty, or we could conclude that the increase in TOC was a naturally occurring process. In order to determine why the concentrations of TOC varied during different times of the year, a more comprehensive evaluation would be necessary to prove it is a natural phenomenon.

RECCOMENDATIONS

1. The current setback of 50 ft from streams, as required by North Carolina regulations, appears to be adequate and should be maintained.

2. The location of septic system drainfields on the landscape should consider the potential subsurface flow from upslope areas, and divert runoff from paved or other impermeable areas around the dwelling.

3. Care should be taken to properly design, install, and manage septic systems installed in the vicinity of streams and other surface water bodies.

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Estimating Absorption Width & Mounding with Your Soil Information.

David Gustafson, University of Minnesota

ABSTRACT

The design of septic system soil treatment areas (STA) includes the evaluation of two important factors, absorption width and groundwater mounding. The introduction of additional risks such as reduced vertical separation, increased loading rates, or poor soil conditions warrants a more complete evaluation of these two factors that could impact a site's ability to perform effective wastewater treatment. This paper introduces a straight forward method in which a system designer can use familiar concepts to produce conservative estimates of absorption width and groundwater mounding that will inform the design process and help mitigate the lateral impact and groundwater contamination risks of placing septic systems in less than ideal site conditions.

Site Evaluation and System Design Strategies for Severe Sites: Overview of Loading Rates

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ABSTRACT

Soil based onsite wastewater treatment and disposal sites for single-family homes often include challenges not addressed in prescriptive guidelines. The site evaluator / soil scientist / designer must consider the attributes and limitations of a site in formulating a successful design strategy. The site evaluation process requires a considerable amount of expertise. The purpose of a site evaluation is to collect sufficient information to enable the evaluator to 1) understand the soil system and the hydrology of the site, 2) predict wastewater flow through the soil and into the underlying subsurface material, and 3) recommend a preliminarily onsite system design (size / elevation / geometry of infiltrative surface, loading rate, etc.) that compliments the soil and site conditions. A review of boundary identification and assessment, and mass loadings to the boundaries is presented for utilization by the site evaluator.

Increasingly, jurisdictions have adopted a “performance” based approach that, in some cases, has been incorporated into regulation. The regulation typically addresses the evaluation, design, and permitting of individual onsite wastewater treatment and dispersal systems for single family homes or small flows application. Acceptable standards regarding soil conditions, treatment quality, and method of dispersal are established. These requirements are not prescriptive “deem to comply” or perform. (NC DHHS, 2006; VDH, 2012) Additional guidance may or may not be provided to the site evaluator and system designer to justify anticipated acceptable performance of a site specific treatment works in compliance with the public health and environmental protection goals of the regulation.

With the relatively recent advent of pretreatment technology, advanced soil dispersal methodologies, and the need to address soils with previously regarded unsuitable conditions, there is consideration of soil receiver sites with “multiple” minimum depths to limiting features. Characteristics may include seasonally high water tables, permeability / flow restrictive features, and other site-specific limiting conditions.

Some 50% of onsite systems are greater than 30 years old (EPA 2002, 1997 census data), many sited, permitted, and installed under little or no criteria and are not compliant with current requirements, which necessitate special design considerations. In these repairs, soil characteristics, area available, disturbed soils, and horizontal offsets are typical limitations.

Soils considered for application of “alternative systems” are now for the most part Moderately Well Drained to Somewhat Poorly Drained. In the case of pretreated effluents, in addition to reduced soil remediation thickness, absorption area loading rates may be allowed to increase substantially.

MATERIAL AND METHODS

Review of Mass Loadings and utilization of the boundary design methodology as outlined in of the *On-Site Wastewater Treatment Systems Manual*, Chapters 4 and 5, is the general format. Traditional infiltrative surface loading rates and area (hydraulic footprint) loading sizing are reviewed. Instantaneous loading rates will be discussed as a tool to properly address a site's characteristics. The concept of hydraulic linear loading will be discussed. Supplemental information from additional sources is cited and referenced.

DISCUSSION

Virtually all research involving the use of septic tank or pretreated effluent into the soil involves controlled application, whether onto columns or in situ, typically with low pressure distribution or drip dispersal. To fully apply the concept of boundary design by anticipating mass loadings, in this discussion equal distribution to the entire absorption area is assumed and necessary.

Site Delineation A complete site evaluation includes a surface characterization of topographic features and horizontal setbacks, a subsurface (soil) evaluation, and the accurate delineation of the soil adsorption area. The delineation is best performed by the site evaluator. Care should be exercised to insure accuracy particularly in the case of sites with limited area, complex topography, and verification of available area. ASTM D 5925-96 provides excellent guidance regarding the elements of site delineation.

<u>Design Boundaries</u>

Each site has multiple design boundaries, not all will control design. Not all boundaries are barriers. Transformations will occur and should be anticipated (OTIS, 1984).

The *infiltration surface* is where wastewater first contacts the soil and is traditionally the (only) regulatory prescriptive loading. Utilization of pretreatment prior to soil application changes the dynamic of the infiltrative surface. The soil infiltrative surface may be preceded by elevated sand bed pretreatment. The sand bed becomes the first boundary followed by the sand / soil interface boundary, a secondary infiltrative surface (EPA, 2002), and a very critical component of design and installation.

Secondary infiltration surfaces are located beneath the infiltration surface layer. Within a few centimeters from the point of application, the STE behaves as water. Additional boundaries due to texture, structure, consistence, restrictions etc. affect vertical movement increasing saturated flow (vertical and horizontal) in response to the boundary. The infiltrative surface and the vadose essentially act as fixed film bioreactors, providing dispersal and treatment of the applied wastewater (EPA 2002).

The *receiving environment* is where renovated effluent is discharged to the ground or surface water system. Constituents of concern may include fecal coliforms, nitrogen, and emerging contaminants.

Hydraulic Mass Loadings / *Daily*

Daily mass loadings are given in gallons per day per square foot of boundary surface area and typically apply to the primary infiltration surface (EPA, 2002). Hydraulic loading rate is the quantity of water applied to a given treatment component, usually expressed as volume per unit of infiltrative surface area per unit time, e.g., gallons per day per square foot (gpd/ ft^2) (CIDWT, 2007).

Traditionally gpd/ ft^2 loading rates are applied to standard trenches 2-3' wide. Absorption area required is based on percolation tests and / or soil texture. Loading rates are applicable to Septic Tank Effluent (STE). Trench bottom loading rates for STE trenches were initially empirically derived based on past experience, and have undergone extensive scientific verification. For the most part they are similar for all states.

Loading rates are often estimated by assigning textures to the USDA four soil groups of Sands, Loams, Fine Loams, and Clays and expressing the loading rates as ranges.

Assignment of loading rates has progressed from less emphasis on infiltration testing to a morphological evaluation considering texture, structure, consistence, clay mineralogy, etc. as evidenced in the current EPA loading rate chart (EPA, 2002). Note that in the chart, the range of STE loading rates reflects the original EPA (1980) loading rates. There is an increase in the loading rate for moderate and strong structure.

Loading rates may also be increased for the reduction of organic loading, BOD_5 by additional pretreatment. Reduction in the biological demand may allow for a reduction in infiltrative surface (Tyler / Converse, EPA 2001, Siegrist 1997).

Where infiltrative surfaces for STE are typically loaded at 2-12% of saturated hydraulic conductivity (K_{SAT}) for sands to clay respectively, higher percentages of K_{SAT} may be appropriate with pretreated effluent and proper design (NC DHHS, 2013;Tyler and Converse, 1984; EPA, 2002):

> "Wastewater infiltration systems sized to receive highly pretreated effluents have a greater risk of failure due to rapid development of a severe clogging mat if the pretreatment unit fails and delivers low quality wastewater to the soil. Also, high loading rates lead to reduced wastewater retention time in the soil, reducing the treatment of wastewater polluntants and allowing pollutants, such as coliforms to move outside of the treatment boundaries of the system" (Tyler and Converse, 1984).

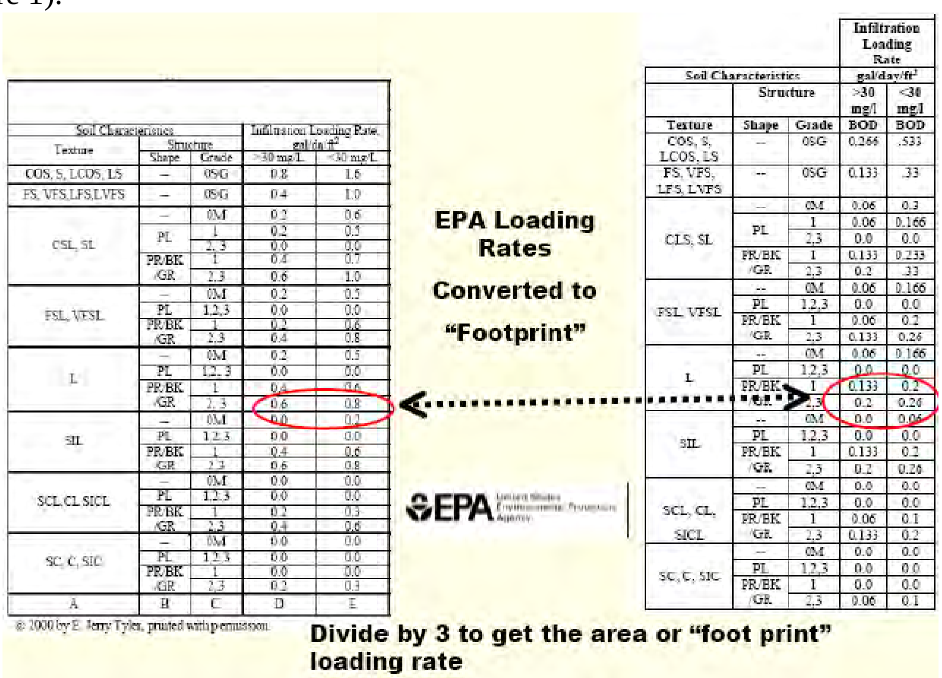
Siegrist (1987) states that hydraulic loading rates to infiltrative surfaces may be increased with pretreated effluents. A table exhibiting loading rates for various wastewater strengths is offered. However, it is further stated "The only limitation on hydraulic loading rate would be the saturated hydraulic conductivity properties of the natural soil. To maintain low moisture contents and adequate soil aeration, the hydraulic rate should remain well below the saturated hydraulic conductivities of the soil (e.g. only

3 – 5% of the K_{SAT})”. Current practices provide for considerably higher K_{SAT} percentages (NC DHHS, 2006; NC DHHS, 2013; VDH, 2012).

Gravity dispersal of pretreated effluent to conventional trench type systems is an inappropriate practice for several reasons. Pretreated effluent will not form a biomat at the trench bottom interface, and effluent will readily flow deeper into the soil column. Without the protection of the soil by the anaerobic biomat there is the potential for the translocation of fines deeper in the soil column that may cause soil clogging. Under increased loading rates, the biological “gluing” agents, an important component of soil structure, may be washed away. When the hydraulic conductivity of the soil is exceeded as would be expected at the front end of a gravity flow trench, the pores are full and the effluent moves down the trench. When circumstances are such that pretreated effluent “creeps” down the trench, the entire soil column, not just the surface, is clogged, and will likely become anaerobic. Resting will likely not restore the soil once this condition occurs.

Areal loading rate is the quantity of effluent applied to the footprint of the soil treatment area (or the absorption area of an above-grade soil treatment area) expressed as volume per area per unit time, e.g., gallons per day per square foot (gpd/sq. ft.) (CIDWT, 2007).

Traditional trench systems are typically installed on centers three times the width of the trench. The resultant area, the footprint is the “areal” loading rate. The “areal” loading rate is approximately one third of the infiltrative surface (trench bottom) loading rate (Figure 1).



Soil Characteristics			Infiltration Loading Rate, gal/da/ft²	
Texture	Structure Shape	Grade	>30 mg/L	<30 mg/L
COS, S, LCOS, LS	—	0SG	0.8	1.6
FS, VFS, LFS, LVFS	—	0SG	0.4	1.0
CSL, SL	—	0M	0.2	0.6
	PL	1	0.2	0.5
		2, 3	0.0	0.0
	PR/BK /GR	1	0.4	0.7
		2,3	0.6	1.0
FSL, VFSL	—	0M	0.2	0.5
	PL	1,2,3	0.0	0.0
	PR/BK /GR	1	0.2	0.6
		2,3	0.4	0.8
L	—	0M	0.2	0.5
	PL	1,2, 3	0.0	0.0
	PR/BK /GR	1	0.4	0.6
		2, 3	0.6	0.8
SIL	—	0M	0.0	0.2
	PL	1,2,3	0.0	0.0
	PR/BK /GR	1	0.4	0.6
		2,3	0.6	0.8
SCL,CL,SICL	—	0M	0.0	0.0
	PL	1,2,3	0.0	0.0
	PR/BK /GR	1	0.2	0.3
		2,3	0.4	0.6
SC, C, SIC	—	0M	0.0	0.0
	PL	1,2,3	0.0	0.0
	PR/BK /GR	1	0.0	0.0
		2,3	0.2	0.3
A	B	C	D	E

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Soil Characteristics			Infiltration Loading Rate gal/day/ft²	
	Structure		>30 mg/l	<30 mg/l
Texture	Shape	Grade	BOD	BOD
COS, S, LCOS, LS	—	0SG	0.266	.533
FS, VFS, LFS, LVFS	--	0SG	0.133	.33
CLS, SL	—	0M	0.06	0.3
	PL	1	0.06	0.166
		2,3	0.0	0.0
	PR/BK /GR	1	0.133	0.233
		2,3	0.2	.33
FSL, VFSL	--	0M	0.06	0.166
	PL	1,2,3	0.0	0.0
	PR/BK /GR	1	0.06	0.2
		2,3	0.133	0.26
L	--	0M	0.06	0.166
	PL	1,2,3	0.0	0.0
	PR/BK /GR	1	0.133	0.2
		2,3	0.2	0.26
SIL	--	0M	0.0	0.06
	PL	1,2,3	0.0	0.0
	PR/BK /GR	1	0.133	0.2
		2,3	0.2	0.26
SCL, CL, SICL	—	0M	0.0	0.0
	PL	1,2,3	0.0	0.0
	PR/BK /GR	1	0.06	0.1
		2,3	0.133	0.2
SC, C, SIC	—	0M	0.0	0.0
	PL	1,2,3	0.0	0.0
	PR/BK /GR	1	0.0	0.0
		2,3	0.06	0.1

Figure 1. EPA (2002) infiltrative and areal loading rates for STE and pretreated effluent.

Areal loading rates represent the area that needs to be delineated for a trench or trench area equivalent system. Areal loading rates are important when characterizing and evaluating a restrictive feature, (secondary boundary), below the primary point of infiltration. The analysis may assume the higher values of saturated hydraulic conductivity, that is restrictive features may be loaded at higher K_{sat} percentages with judgment assuming saturated leakage (Amoozegar, 2004). These same boundaries and saturated flow come in to consideration when addressing contour loading.

Hydraulic Mass Loadings / *Instantaneous*

Instantaneous mass loadings are given in gallons per dose per square foot of boundary area (EPA, 2002).

The application of effluent at an appropriate instantaneous dose is achievable through engineering design. Input from the site evaluator is important to develop an appropriate design.

In the case of soil based wastewater treatment and disposal, dispersal design must provide for adequate reaeration of the soil body to maintain an efficient, sustainable system. Encouraging unsaturated flow, without significant infiltrative surface clogging, increases residence time and insures that the larger, macropores remain open for a majority of the time. The necessary diffusion of oxygen will maintain an aerobic treatment environment. The most efficient application is to provide light, uniform dosing of the soil.

In practice there are two types of distribution systems utilized in the application of effluent to the subsurface; Low Pressure Distribution (LPD) and Drip Dispersal.

LPD systems are applied in conventional gravel (or similar leach products) trenches or beds. The networks consist of solid PVC pipe manifolds that supply water to a series of smaller perforated PVC laterals. The laterals are designed to discharge nearly equal volumes of wastewater from each orifice in the network when fully pressurized. This is accomplished by maintaining a uniform pressure throughout the network during dosing (EPA 2002). In the case of LPD systems the instantaneous dose maybe expressed as

gallons per orifice per dose, gallons per dose per ft^2 trench bottom, and inches per dose trench bottom. Each orifice represents a point load. Recommended orifice maximum density is one per 6 ft^2 of trench bottom infiltrative surface loading to provide equal distribution and maximum potential for treatment. In trenches, dose frequency is typically less than two doses per day.

Wastewater drip dispersal networks provide uniform distribution of preconditioned wastewater over infiltrative surfaces of land application systems. The unique feature of drip dispersal networks is the use of uniformly spaced drip emitters that are inserted within flexible tubing to control the rate of wastewater discharges out of the tubing through small orifices. Typically, the dripperline is installed directly into the soil (NOWRA, 2006).

In the case of Drip Dispersal systems, the instantaneous dose may be expressed in gallons per emitter per dose, and gallons per linear foot of tubing per dose. Orifices are typically spaced two feet along the tubing with the tubing installed in soil two feet apart for a maximum orifice density of one per 4 ft^2 of the areal loading. Dose frequency is typically four doses per day often in two or more zones.

The following is a comparison summary of Drip Dispersal with LPD.

•Direct burial vs. trenches, shallow placed <12”, minimal soil / site disturbance
•Area loading (gal./$ft.^2$) approximately equal to LPD Systems
•Linear loading (gal./feet length of pipe)
  –2 to 3 times less
  –Smaller centers
•5 to 10 times more orifices
•Flows < 20% per orifice (.73 – 1.3 gal/min per orifice at 3’ head vs. 0.01 gal/min per emitter)
•Doses 10 to 20% of LPD volume

Micro Dosing takes maximum advantage of porosity of the soil to increase residence time, maintaining a primarily aerobic environment for BOD (organic pollutants) reduction, bacteria and virus die off, and chemical attenuation of nutrients in the soil. With the tubing in the soil, effluent is efficiently recycled into the shallow soil biological system. The concerns over soil clogging of the infiltration surface due to the organic load, suspended solids, biomass, and other products from organisms living on the wastewater is minimal when compared to traditional constructed infiltrative surfaces (trenches and beds).

In Emerick et al. (1999), three sand filters were dosed in a controlled manner. A summary analysis of volume per dose for various hydraulic loading rates and dose frequencies was generated in a table for the three filters at three loading rates. As the loading rates increased, there was a need to increase frequency of dosing, keeping the same instantaneous (volume per dose) in order to maintain a desirable percentage of field capacity providing for open macro pores. Higher percentages result in more saturated conditions and deeper penetration into the filters.

In the case of a drip dispersal placed in a high clay content soil, an instantaneous dose of 0.075 - 0.1 gallon per emitter per dose is appropriate, where in the case of a sandy loam texture an instantaneous dose of up to .2 gallons per emitter per dose may be proper, depending upon the type, extent, and distribution of pores.

According to TVA (2004):

"Reducing both the daily and instantaneous hydraulic loading rates and providing uniform distribution over the infiltration surface can help maintain lower soil moisture levels. Lower soil moisture results in longer wastewater retention times in the soil and causes the wastewater to flow though the smaller soil pores in the unsaturated zone, both of which enhance treatment and can reduce the necessary separation distance."

In Long (1995), an exhaustive literature review was conducted regarding Nitrogen Dynamics in conventional on-site septic tank effluent absorption systems. The elements of instantaneous dosing were emphasized. Equal distribution was assumed. Several natural mechanisms were identified. Within the soil column, microbial biofilm "micro sites" on soil aggregates optimize nitrogen removal with alternating aerobic / anoxic environments. As well, unsaturated flow, increased residence time, low velocity "film" flow, and reduced hydraulic loading creates conditions for nitrogen removal.

Similar conclusions were expanded upon in chapter 2, section 2.3 (specific to Nitrogen) of the more recent WERF reports by McCray (2009 and 2010).

The reaction rates of nitrogen compounds in soil are generally rapid. Many reactions occur within minutes in the appropriate conditions. The reactants must be available and the soil environmental conditions must be acceptable. By way of soil texture, structure, organic matter content, and porosity, shallow soils have the range of moisture status conditions within the medium providing the environment necessary for the processes of nitrification and denitrification. These elements include dynamic aerobic, facultative, and anaerobic conditions in close proximity. The major limitation for nitrification is insufficient oxygen. Soil air O_2 in shallow soils is adequate for nitrification. The major limitation in denitrification is organic carbon deficiency. Surface horizons have the highest amount of organic carbon available. STE provides an additional source of organic carbon.

Conditions beneficial to denitrification occur when the soil pores are at least 60% saturated, or when the soil air contains no more than 10% oxygen. Denitrification may occur also in well-aerated soils, in anaerobic micro sites ("hot spots"). Soil column biofilm "micro sites" provide variable moisture and oxygen status. A low fluid velocity environment is conducive to the formation of facultative biofilms that are very stable. (McCray, 2009)

The dual (air and water-filled) soil porosity dynamic is maximized with drip dispersal. Drip dispersal provides distribution across the entire absorption area with a high (minimum one emitter per 4 ft^2) orifice density, and low *linear* loading, insuring maximum soil contact. Each point source application volume is equal. Instantaneous doses are soil characteristic dependent (.075 - .2 gallons per orifice per dose, over 5 – 20 minutes) providing for pore saturation during and just following application (denitrification) followed by unsaturated flow and nitrification between dose cycles.

Shallow placed drip dispersal has been assigned a 50% nitrogen reduction credit when applied to the soil component in Group II-IV soils. (Tetra Tech 2013, EPA)

Another engineering concept related to instantaneous dosing is flow equalization and time dosing. Timed dosing is a configuration in which a specific volume of effluent is delivered to a component based upon a prescribed interval, regardless of facility water use; (CIDWT, 2007). Flow equalization is a configuration that includes sufficient effluent storage capacity to allow for uniform flow to a subsequent component despite variable flow from the source (CIDWT, 2007)

The peak daily design flow is usually the prescriptive basis for absorption area sizing. Average flow typically represents 50 – 80% of the peak daily design flow.

Hydraulic Mass Loadings / *Linear*

Linear mass loading are given in gallons per day per foot along the boundary surface contour (EPA, 2002).

The hydraulic linear loading has its basis in Darcy's Law considering the higher values of lateral saturated hydraulic conductivity in response to slope over a boundary feature. There are five fundamental models to apply as appropriate to an individual site (i) Creviced Bedrock (vertical flow), (ii) Apparent Water table (mounding), (iii) Semi permeable Layer (Leakage), and (iv) Impermeable Layer (horizontal flow) (Converse and Tyler, 2000). The models exhibit vertical and horizontal response to a horizontal boundary feature.

Tyler (2001) has developed a chart that is a helpful tool of ascertaining the appropriateness of a given site's contour loading assigning values based on peak daily design flows as a function of soil texture, soil depth, and slope. The effect of lateral flow is minimized by extending the system contour as the site allows. Landscape linear loading requirements are based on site / soil determination (estimation) of vertical and horizontal subsurface water movement over and through the limitations and will vary with conditions including slope, texture, and depth to limitation.

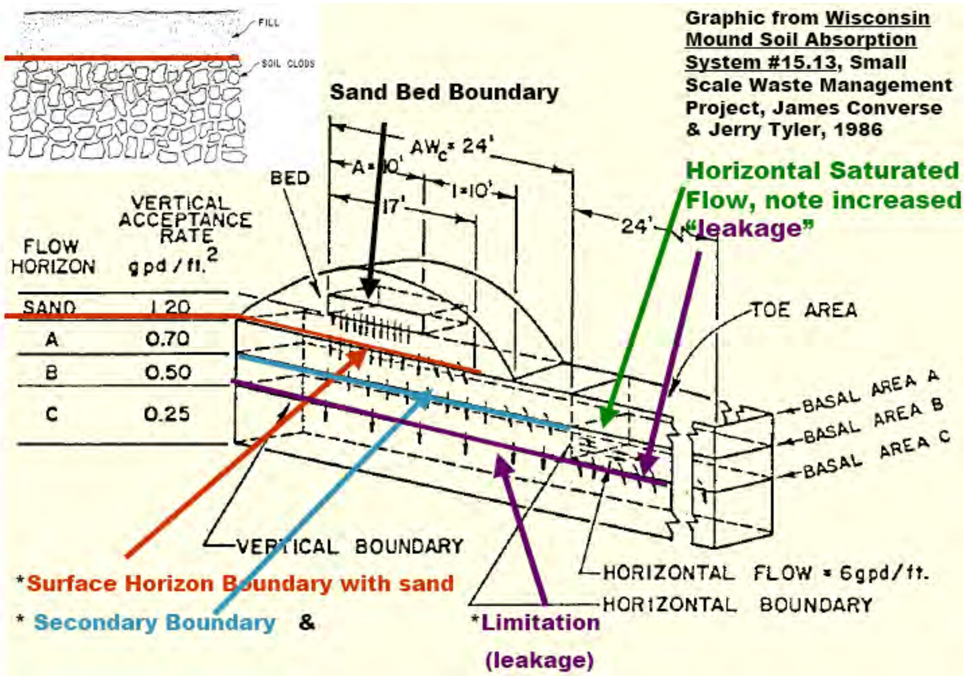

Figure 2. Summary graphic of boundaries and mass loading (Converse / Tyler, 1986).

In a detailed lateral flow analysis appropriate additional information would include the K_{sat} of the zone just above the restriction (saturated lateral flow), and the restriction (saturated leakage). Flow equalization and time dosing may allow analysis based on average flows.

For an in ground system, such as a drip dispersal system accommodating a need for shallow installation, sited with the appropriate areal loading rate, the system size is potentially three times larger than the above ground basal area sized system. The larger size encourages an unsaturated flow system with a larger amount of area for saturated leakage through the restriction.

CONCLUSIONS

The soil treatment system is integral and is the site-specific component of an onsite system. A site evaluation must delineate the absorption area, ascertain site limitations and attributes, characterize their extent, and offer design approaches for system success. The site and soil evaluation process follows a systematic approach that includes delineation of the absorption area, describing the surface characteristics, and subsurface characteristics, identifying the attributes and limitations and interpreting their relationship for use in an onsite system design. The process of evaluation and design may often be repeated several times for each proposed site. During each repetition, new information is obtained and a new design approach is considered. This process continues until a system design recommendation is developed that provides the optimal match to the site conditions observed. The evaluation should communicate to the designer site-specific details of the delineated area and include a preliminary system layout and design. The report should address site specific conditions such as soil quality, slope, stoniness, vegetation, surface drainage, site preparation, depth of installation, etc. that may, in the judgment of the evaluator, affect the design and/or field installation.

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Determining the Minimum Subsoil Permeability for Pressurised Infiltration Systems for on-Site Wastewater Treatment in Ireland.

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ABSTRACT

In Ireland approximately one third of rural households depend on onsite wastewater treatment systems for their effluent disposal (CSO, 2011). The effectiveness of these systems in the prevention of water pollution from microbiological pathogens and/or nutrients is dependent on the thickness and permeability of the unsaturated layer of overlying subsoil receiving the effluent (i.e. the percolation area). In Ireland, glacial till (low permeability boulder clay) is the most widespread subsoil covering over 43% of the country area at surface, and approximately 25% beneath peat or floodplain sediments (Meehan & Lee, 2012) and is often unsuitable for traditional on-site treatment systems.

This research has evaluated two alternative technologies to the conventional gravel-filled gravity fed percolation trenches in low permeability Irish subsoils: drip distribution systems (DDS) and low pressure pipe systems (LPPS). Two sites in Ireland were upgraded from traditional septic tank soakaway systems to the two alternative pressurised infiltration systems in parallel. At each site, 50% of the effluent was distributed via a LPPS, the other 50% via a DDS. The three-dimensional distribution of the hydraulic load and resulting soil moisture profiles within the percolation area beneath each system across the seasons were monitored through the installation of tensiometers and soil moisture probes across the area and at a range of depths from the upper soil horizons down to the water tables. The timing and volume of flows to the percolation areas were recorded as well as rainfall and other meteorological data in order to understand the effectiveness of the systems across the year in the Irish climate.

Both infiltration systems had been designed to the same areal loading rate at ~ 3 $L/m^2.d$ and over a two year monitoring period the soil moisture results under both the DDS and the LPPS have shown that the low permeability subsoils beneath the systems maintained unsaturated conditions and with no evidence of surface ponding. Water quality samples have also been taken of the percolating effluent in the vadose zone and downstream groundwater which have revealed excellent attenuation of on-site effluent contaminants within 1 m depth underneath the infiltration surface. The soil moisture results have then been used to calibrate a HYDRUS 2D model of the unsaturated percolation areas. These models have then been used to predict DDS and the LPPS performances on even lower permeability subsoils in order to establish design criteria limits for both the LPPS and DDS. Hence, minimum subsoil permeabilities based on the Irish falling head percolation test have now been defined for the suitability of such pressurised infiltration systems which will augment the legislative Code of Practice for Single houses in Ireland.

Expected Treatment Level in a Soil Based Treatment System.

Dennis Hallahan, Infiltrator Systems Inc.

ABSTRACT

SSSA Onsite Wastewater Conference, Innovation in Soil-Based Onsite Wastewater Treatment April 7-8, 2014, Albuquerque, NM Session Selection: Design and Evaluation of Pretreatment Technology Title of Paper: Soil Based Treatment Abstract Infiltrator Systems Inc. has research ongoing; the research should be completed prior to the conference but will not be complete at time of Abstract submittal, September 30, 2013. Until the research is completed it is confidential. I spoke to Mr. Buchanan and he mentioned that it would be acceptable to propose a presentation on the research as this type of problem with timing is common in the research environment. If there is a problem, such as the research is delayed, then as a substitute I have the following abstract that could be utilized: Presentation: Onsite Wastewater Treatment System Drainfield Malfunction: Causes, Investigation, Prevention, and Correction Abstract: The lifespan of an onsite wastewater treatment system drainfield is influenced by numerous factors, including siting, vertical separation distance, maintenance, wastewater flow volume, septic tank volume, as well as other factors. The presentation will review methodologies to diagnose problem site systems. The intention is to have the presentation serve as a learning tool on the potential causes, how to investigate and once the problem is understood then recommending a proper solution. The presentation will review: Malfunction investigation basics, septic tank investigation, function of the tank, drainfield investigation, and malfunction issues and examples. About the Presenter: Dennis F. Hallahan, PE Mr. Hallahan has over twenty five years of experience with onsite wastewater treatment systems' design and construction. He has authored several articles for onsite industry magazines and has given numerous presentations nationally on the science and fundamentals of onsite wastewater treatment systems. Dennis is currently Technical Director at Infiltrator Systems, where he is responsible for research and technology transfer between Infiltrator Systems and the regulatory and design communities. Dennis also oversees a staff that is responsible for product research and testing for both universities and private consultants. Dennis also assists in the design and specification of large decentralized systems, many exceeding 1 MGD. He received his MS in civil engineering from the University of Connecticut and his BS in civil engineering from the University of Vermont. Dennis is a registered professional engineer in Connecticut. Dennis also holds several patents for on-site wastewater products. Contact info: Dennis F. Hallahan, P.E. 4 Business Park Rd. Old Saybrook, CT 06475 P: 800.221.4436 dhallahan@infiltratorsystems.net

Measuring In Situ Saturated Hydraulic Conductivity (Ksat) Using the Automated Aardvark Permeameter

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ABSTRACT

Onsite wastewater disposal systems are a commonly used practice in Georgia. The Georgia Environmental Health Department regulates this practice. Design of systems is based upon observed soil characteristics. The investigating soil scientist, engineer or geologist provides an Estimated Percolation Rate dependent on observed soil characteristics for most systems. In the case of an advanced system or special situation, percolation (or Ksat) is measured in the field. The common measurement methods used are often cumbersome or fragile. The Aardvark Soils Borehole Permeameter is designed to be rugged and , transportable, with the option for and optionally automationed. First used in the field ten years ago, this method has proven to be reliable. In 2010 Soilmoisture Inc. joined with Aardvark Systems International to produce an automated version (with manual options) that greatly increases production rates and report capability. Testing at The University of Georgia's new Department of Crop and Soil Sciences Department research farm, The Iron Horse Farm, is successfully, and quickly, acquiring design data for onsite wastewater disposal, irrigation and tile drainage. Testing for this study is concentrated on onsite wastewater disposal systems where type and size are chosen based on Ksat rates.

INTRODUCTION

The University of Georgia's Department of Crop and Soil Sciences has purchased a new research farm located in Green and Oconee Counties, GA. The farm has a mixture of cropped and forested land. It is situated on the banks of the Oconee River, and the landforms are a mixture of recent alluvial, terraced alluvial, and residual parent materials. Drainage ranges from wetlands to well-drained soils, and soil textures range from sandy to clayey surface textures and control sections. Soil pedon depths range from 0 to more than 1.5 m. The original USDA-NRCS Soil Survey of the land was conducted at a scale unsuitable for research purposes. The University of Georgia selected this farm after an exhaustive study of similar candidates in the area. Potentially highly productive soils, available water sources, and proximity to the university made this the best choice. Over the last year an intensive natural resource inventory has identified the presence of many soils that were not be mapped at the NRCS Soil Survey level. Management will require correctly designed irrigation and tile drainage systems for optimal farm use. The building of these facilities necessitates onsite wastewater disposal systems. The university, through Aardvark Systems International (Thomas Macfie), is mapping Ksat at the farm for many uses. Time is limited, since the research farm needs to be in full use in 2016; therefore a quick and accurate method for determination of Ksat is required.

MATERIALS AND METHODS

Among the soil series being investigated for wastewater treatment at the farm are the Altavista, Whistlestop and Wickham Series. These series differ considerably, and may require different systems due to differences in water table, impermeable layers, or parent material. Typical onsite wastewater treatment systems in Georgia are trench with or without aerobic pretreatment, Elgen and drip systems.

The Altavista series is fine-loamy, mixed, semiactive, thermic Aquic Hapludults. The A horizons are typically 0.3 m of light brown sandy loam. The upper Bt is a sandy clay loam to about 0.9 m deep. The lower Bt or Btg is a gray sandy clay loam to about 1.5 m deep. The Altavista pedon studied is no longer in a floodplain due to a water control dike which separates this area form the Oconee River.

The Whistlestop Series is fine-mixed, thermic Oxyaquic Kanhapludults. The A horizons are typically 0.2 m of reddish brown sandy loam. The upper Bt is a red clay or clay loam to about 1.2 m deep. The lower Bt or Btx is a red, yellow and gray, platy structured clay or clay loam to about 2 m deep. The Whistlestop pedon studied is in a high terrace position about 20 m above the Oconee Rive and does not flood. The soil is often adjacent to residuum (Cecil and Madison series). The Whistlestop soils resemble the Cataula soils formed in residuum or colluvium.

The Wickham series is a fine-loamy, mixed, semiactive, thermic Typic Hapludults. The A horizons are typically 0.3 m of reddish brown sandy loam. The Bt is yellowish or reddish or a mixture of the two hues throughout the soil pedon. The thickness of the pedons varies from 0.5 m to more than 1 m depending on the extent of soil development. The Wickham pedon studied is 10 m above the Oconee River on an alluvial terrace that does not flood.

Hydraulic saturated conductivity (Ksat) is a required design variable to determine trench depth and spacing of systems. Whether expressed as percolation rate, infiltration rate, or Ksat, solute movement in soil can be expressed in different units. The Aardvark Borehole Permeameter expresses each of these variables depending upon choices which the operator makes. In this study the unit chosen is Ksat in cm/h. However, Ksat can be reported in inches/h or other ratesdepending on the needs of the report. Typically, in tThe United States the most common unit usedterm is inches/hour. In Georgia, the Environmental Health Department uses the percolation rate of minutes/inch for onsite wastewater system design.

At each of the three research sites, an individual set up a group of nine Aardvark Permeameters to run simultaneously, with three permeameters running at three different depths. Using a 8 cm (3 ¾ inch) bucket auger, holes were bored to depths of 0.3, 0.6 and 0.9 m in the Altavista site. No holes were bored deeper due to an existing seasonal high water table during the study period. In the Whistlestop and Wickham soils, holes were bored to 0.3 m, 0.6, 0.9, 1.2, 1.5 and 1.8 m (1 foot, 2 feet, 3 feet, 4 feet, 5 feet and 6 feet). The Aardvark Ground Units ran for 15 min for initial saturation. The above-ground water reservoirs are weighed in tenths of a gram (one gram is approximately one milliliter) every minute after the initial saturation until steady state is attained. A consistent rate, varying by no more than 10%, for four consecutive reading means steady state has been achieved. This measurement was repeated in three representative sites for each soil series, resulting in 9 readings per depth per series. Typically it took from 30 min to 1 h to reach steady state. The time period of the reading can be set from 12 s to days. Very short time periods results in apparatus and soil noise, while very long reading periods lengthens the time needed to study each borehole.

RESULTS AND DISCUSSION

Aardvark Permeameter readings, simultaneously using 9 permeameters, were manually conducted at a rate of 27 sites (?) per 6-8 h period by a one-person team. This could be increased dramatically using two persons and an automated group of permeameters. The results of these measurements are shown in Table 1.

Table 1. Ksat values for the three soil series at the University of Georgia farm. Percolation rate values, in min/inch, are given in parentheses.

			Ksat (inches/hour)			
Soil Series	**0.3m**	**0.6m**	**0.9m**	**1.2m**	**1.5m**	**1.8m**
Altavista	1.42 (4)	0.53 (12)	0.03 (204)	-	-	-

Whistlestop	1.56 (3)	0.67 (13)	0.12 (51)	0.03 (204)	0.02 (300)	0.02 (300)
Wickham	1.83 (2)	1.01 (6)	0.45 (14)	0.40 (16)	0.33 (24)	0.33 (24)

With respect or Georgia guidelines the following systems are suitable as described. Based upon Georgia Environmental Health Guidelines, the Altavista soils are suitable for a drip system and possibly a passive aerobic Elgen system based upon further study at 0.45 m. The Whistlestop soils are suitable for a drip system, shallow aerobic and an Elgin System. The Wickham soils are suitable for conventional onsite wastewater treatment systems of trenched gravel or manufactured products substituting for gravel. The Wickham soils are by far the most economical soils to use for onsite wastewater treatment and disposal. Estimated typical costs for a three bedroom house onsite system would be $4,000 for conventional, $8,000 for aerobic, and $16,000 for a drip systems as of 2014

CONCLUSIONS

The data rapidly and consistently collected with the Aardvark Permeameters can be used for several applications. The Altavista soils studied are designated for tile drainage. The Whistlestop soils are to be used for plant breeding and will do well under irrigation. The Wickham soils are the most versatile, and are well suited for many uses, including onsite wastewater treatment.

When collecting data, reliability of the equipment is as important as other factors. A user would be very disappointed to drive two hours each way, bore multiple holes, and then have the instruments break due to fragile construction or ill design. We have found that, in the several years of using the Aardvark Permeameters, they have more than met our rigorous standard of reliability

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An Investigation for the Need of Secondary Treatment of Residential Wastewater when Applied with a Subsurface Drip Irrigation System

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ABSTRACT

Two subsurface drip irrigation (SDI) systems were installed and monitored at two sites in Tennessee. These locations were residential developments each served by a septic tank effluent pump (STEP) collection system, a recirculating media filter (fine gravel media), and SDI dispersal. At both locations, SDI research plots were established to receive primary treated (septic tank effluent) and secondary treated (recirculating media filter effluent) wastewater. In close proximity to randomly selected SDI emitters, soil samples were extracted. Soil cores were analyzed to determine hydraulic conductivity, and pore water samples were analyzed for nitrate, total nitrogen, total carbon, and total phosphorus. Results indicate that the primary-treated side had lower hydraulic conductivity values, higher nitrate and higher total nitrogen levels than the secondary-treated side and the background soil. Interestingly, the primary-treated side had less total carbon, and the background phosphorus concentration was twice that of the primary and secondary treated sides. The primary effluent application site showed a decrease in concentration for all constituents with increased depth. Secondary treatment does result in a higher quality effluent but is not needed when applying effluent with a SDI.

Subsurface drip irrigation (SDI) has been widely adopted as an alternative effluent dispersal method for sites with shallow restrictive soil features. Pressurized hydraulic networks ensure uniform effluent distribution across the soil adsorption area. This improves the treatment potential by maximizing the contact of effluent with soil particle surfaces. It is a common practice to provide secondary treatment to wastewater that is to be dispersed via SDI. Pre-treatment is usually provided by an aerobic treatment unit or a packed-bed media filter. This 'requirement' for pre-treatment is largely based on protecting the emitters, the in-line devices that control the effluent emission rate from the drip tubing. The need for secondary pre-treatment is debated because good design and management practices have been shown to protect the emitters by providing effluent filtration and by frequent flushing of the drip tubing. Certainly, providing secondary treatment prior to dispersal takes much of the treatment responsibility away from the soil, especially when SDI is used in soils with shallow restrictive features. However, it is thought that the improvement in application uniformity may overcome the limitations in the soil depth required to renovate the effluent.

The primary hypothesis of this study is that secondary treatment is not needed to adequately purify residential wastewater, when SDI is used. This study will prove or disprove this hypothesis by analyzing the soil and soil solution near and below SDI emitters. Hydraulic conductivity will be used to determine differences in soil physical properties related to the application of secondary-treated and primary-treated effluent. Further, the soil solution will be sampled for nitrate-nitrogen (NO_3^--N), total nitrogen (TN), total phosphorus (TP), and total carbon (TC) to determine differences in water quality beneaths these treatments.

METHODS AND MATERIALS

Site Descriptions

This research was conducted at two residential subdivisions, Jackson Bend (JB), located in Blount County, Tennessee and Crescent Glen (CG), located in Rutherford County, Tennessee. Each subdivision is serviced by a decentralized wastewater management system that consists of a Septic Tank Effluent Pump (STEP) collection system, a recirculating media filter for secondary treatment, and subsurface drip irrigation for effluent dispersion.

At each location, two 93 m^2 SDI research plots were constructed, each with 305 m (1,000 ft) of drip tubing, divided into twenty 15-m (50 ft) laterals. The drip tubing had a nominal diameter of 1.27 cm (0.5 in), and the pressure compensated emitters were rated at 2.3 L hr^{-1} (0.62 gal hr^{-1}). One plot received septic tank effluent (STE) and the other received recirculating sand filter effluent (RSFE). The application rate was 4.1 L m^{-2} (0.1 gal ft^{-2}). Thus, each field receives 757 L (200 gal) per day every day. When sampling began, the reseach plots at CG had been in operation for five years and the JB plots had been in operation for three years.

STE was collected by installing a diversion valve in the effluent sewer just prior to the secondary treatment. The soil in JB is primarily a sandy loam and is 120-240 cm to groundwater. JB is made up of high-end housing with large lots. The soil in CG is primarily a clay loam with about 60 cm to bedrock. The homes in CG are mainly starter homes with small lots.

Sample Collection and Analysis

Four rounds of samples (soil cores), representing four seasons, were taken from each of the four plots. Background soil samples were collected from just ouside of the plots and were used as controls. Soil cores were collected in a similar manner as Jnad (2001a, 2001b). Soil samples were collected with a coring device and transported to the laboratory for analyses. The cores had 5 cm diameters and were 7.5 cm long. Samples were obtained from two depths; 30 cm below the emitter level, and 60 cm below the emitter level. At each depth, samples were collected at six locations relative to the emitter. Each core location was labeled with a number 1-14 depending on its location relative to the drip emitter. Locations 1-12 were located near the emitter while locations 13 and 14 were the control samples (30 and 60 cm depths, respectively). The odd numbered cores correspond with samples taken from the 30 cm depth and even numbered cores correspond with samples taken from the 60 cm depth. Samples 3 and 4 were collected 30 and 60 cm below the drip emitter, other samples were taken 30 cm to either side of the drip lateral, and then 30 cm down the drip lateral. The same pattern was repeated at 60 cm below the drip lateral fig 1). A total of 14 cores were taken from each plot during each sampling event. A much more detailed outline of the sampling procedure can be found at Hillenbrand (2010).

Each boring was initially excavated to a depth of 25 cm so that the coring sampler could extract a sample with the 30 cm depth in the middle of the core. Once the first core was taken, a loose soil sample was collected for soil solution extraction. The hole was then extended to a depth of 56 cm and a core sample and another loose soil sample was collected. The same sampling process was repeated for the control samples.

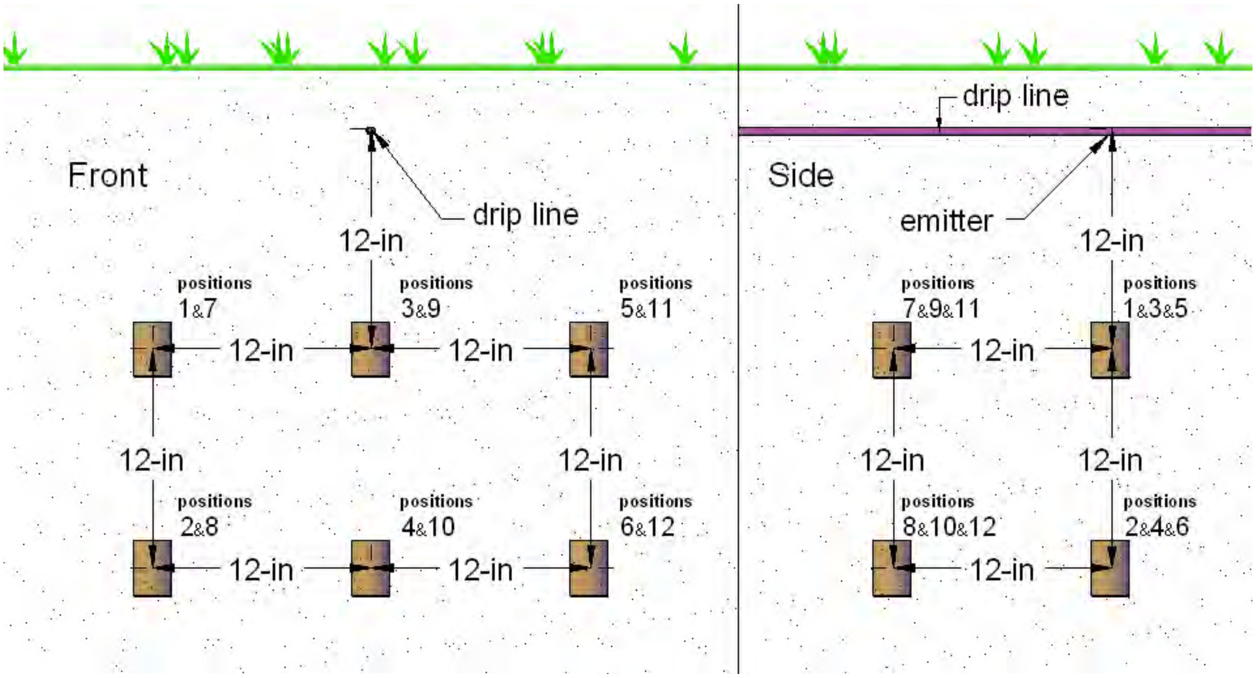

Figure 1. Positions of soil cores relative to drip line and emitter.

Physical Analysis

A falling head permeameter setup was used to determine the saturated hydraulic conductivity (hydraulic conductivity) of each core sample. Fourteen permeameters were installed on a rack to run all 14 samples from each sampling event at one time. Preliminary testing showed that the saturated hydraulic conductivity for the samples ranged from 0.3 to 0.8 cm/day. The cross sectional area in inches of the standpipe (a), the cross sectional area of the sample in inches (A), the length of the sample in inches (L), time in seconds (t), and the heights of the water levels, in inches, relative to the bottom of the sample (H1 and H2) were used to calculate the hydraulic conductivity for each core.

Soil Solution Extraction

Deionized water was used as a solvent to extract the soil solution. Moist soil samples, containing approximately 100 g (dry weight), were added to bottles that contained 50 g of deionized water. Parallel samples were dried at 105°C for 24 hours and weighed to determine the moisture content. This method provided a means to collect a soil solution volume, which could be reliably collected, and the final solution concentration could be corrected for dilution (Klute, 1986). The concentrations are listed on a mg-constituent per kg-soil basis.

Total Kjeldahl phosphorus (TKP) and total Kjeldahl nitrogen (TKN) and soluble nitrogen were determined by the block digestor method (APHA, 2005). Total organic carbon (TOC) was determined using the combustion method (APHA, 2005). TKN measures the organic nitrogen and ammonium in the sample. Soluble nitrogen, which includes nitrate and nitrite, was determined by using the difference in the TKN method and the persulfate oxidation method (APHA, 2005). Most soil elemental analysis does not include soluble nitrogen due to the ease of the TKN method and the concentration of nitrate and nitrite in most soils is very limited. Because wastewater is being applied to this soil, the concentrations of soluble nitrogen should be greater and is important to this study. Nitrogen is reported on an “as N” basis.

Statistical Analysis

The experimental design was a randomized block design – split-plot (RBD-SP). The analysis of variance (ANOVA) using the mixed models procedure for RBD split-plot design (SAS version 8.0, University of Tennessee, Knoxville) was used to analyze the data. Each location (JB and CG) was a whole plot, the split plots were the main treatments (STE, RSFE and Control). Sampling depth became a second factor the split-plot design. The data were blocked on sampling date. Log transformations were performed on Ksat, NO_3^-, TP data, and a square root transformation was used on the TN data. The estimated means reported are the back transformed means. Significance was determined at the 0.05 level. The data are listed in Table 1.

RESULTS AND DISCUSSION

Hydraulic Conductivity

At JB, there was no significant difference in hydraulic conductivity between the STE and RSFE treatments; however the RSFE side did have a significantly lower Ksat than the control. The estimated Ksat values for STE, RSFE, and the Control are as follows: 0.041, 0.036, and 0.073 cm/day respectively. The Ksat differences for 30-cm and 60-cm depths were not significant (0.049 and 0.050 cm/day, respectively).

At CG, there was no significant difference in the hydraulic conductivity between the RSEF and STE (0.042 and 0.027 cm/day respectively). The STE at a depth of 60 cm was significantly lower than the Control at a depth of 30 cm, but not significantly different than RSFE at either 30 cm or 60 cm, or STE at 30 cm.

Nitrate-Nitrogen

At JB, there was no significant difference for nitrate between the STE, RSFE, and Control treatments. Depth was a significant factor for nitrate concentration at JB with the concentration getting higher nearer the emitter (3.970 mg/kg at 30 cm and 2.602 mg/kg at 60 cm). The nitrate concentrations for the RSFE and Control were much lower at CG than at JB, but the CG STE nitrate concentration was nearly twice the JB STE nitrate concentration (11.300 mg/kg and 5.804 mg/kg, respectively). Depth did not matter at CG but the greatest difference in depth occurred with the CG STE samples. The concentration of nitrate at 30 cm below the emitter for the STE side at CG was 14.6 mg/kg, but by 60 cm below the emitter, the concentration was 8.725 mg/kg.

Total Carbon

At JB, TC was lower in the STE as compared to the RSFE and Control treatments but not significant due to the variability in the data. At CG, the TC differences were smaller ranging from 25.6 to 29.8 mg/kg for the STE, RSFE and Control treatments, all of these were less than the RSFE and Control Concentrations from JB. The depth did not seem to impact the concentration of TC found in the soil at either location.

Total Nitrogen

At JB, the TN concentration was not significantly different between the three treatments. The treatment means ranged from 6.4 to 8.4 mg/kg. At JB, the Control at the 30-cm depth had a significantly higher concentration of TN than at the 60-cm depth, but these concentrations were not significantly different compared to the STE and RSFE samples and either depth. The STE-TN concentrations at CG were significantly different from the RSFE and Control concentrations (9.80, 2.80, and 2.81 mg/kg resp.). Depth at CG was not a significant factor.

Total Phosphorus

At JB and CG, there was no significant difference in TP between the three treatments. The control samples from JB at the 30-cm depth were significantly higher than the STE and RSFE samples. The means for RSFE at JB are higher than the means for STE but were not significant. The RSFE samples at CG have a lower TP concentration than the STE and Control samples.

CONCLUSION

The purpose of this study was to evaluate two strengths of wastewater (STE and RSFE) being applied by SDI to determine the need for secondary treatment. The purpose was not to evaluate the performance of SDI as a whole. SDI augments the soil's ability to treat wastewater but its full potential may be diminished by the use of secondary treatment. Physical and chemical properties of the soil were measured to make the comparison. It was found that the pore water in the soil that had been irrigated with the low strength wastewater (RSFE) was of slightly higher quality than the pore water in the STE side. At Jackson Bend, the nitrate-nitrogen, total carbon, total nitrogen, and total phosphorus concentration levels were statistically the same. At Crescent Glen the nitrate-nitrogen and total nitrogen concentration leverls were significantly higher in the STE treated areas but the total carbon and total phosphorus concentration levels showed no significant differences. The benefits of a secondary treatment are not significant enough to make it necessary when using a SDI. The soil provides much of the same treatment as a pre-treatment system, and SDI dispersal systems are designed to fully utilize these characteristics.

Acknowledgments

The authors would like to recognize the following agencies that helped to support this research: Tennessee Department of Agriculture, Nonpoint Source Program; Tennessee Water Resources Research Center, Tennessee Valley Authority, Consolidated Utilities of Rutherford County, and Southeastern Environmental Engineering, LLC.

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Table 1. Modeling results for hydraulic conductivity and soil solution samples taken during investigation. Comparisons are made in three clusters. Cluster 1 – all controls, RSFE, and STE independent of depth; Cluster 2 – all cores at 30 cm vs. all cores at 60 cm independent of treatment; and Cluster 3 – interaction of treatment and depth.

SAS Output for Jackson Bend.

Treatment	Ksat (cm/d)		NO_3^- ppm		TC ppm		TN ppm		TP ppm	
	Estimate	Std Err	Estimate	Std Err	Estimate	Std Err	Estimate	Std Err	Estimate	Std Err
Control	0.073 a	0.019	2.780 a	1.146	42.674 a	13.130	6.428 a	2.861	0.422 a	0.163
RSFE	0.036 b	0.010	1.997 a	0.826	39.132 a	12.564	7.022 a	2.898	0.263 a	0.121
STE	0.041 ab	0.010	5.805 a	2.080	13.560 a	14.016	8.435 a	3.659	0.120 a	0.081
1 (30 cm)	0.049 a	0.012	3.970 a	0.905	33.442 a	10.876	8.603 a	2.655	0.292 a	0.747
2 (60 cm)	0.050 a	0.012	2.602 b	0.629	30.135 a	10.797	6.051 b	2.227	0.214 b	0.063
Control 1	0.068 a	0.024	4.657 a	1.927	43.521 a	13.982	8.947 a	3.579	0.547 a	0.200
Control 2	0.078 a	0.024	1.586 b	0.779	41.826 a	13.433	4.325 b	2.488	0.318 b	0.141
RSFE 1	0.037 a	0.010	2.040 ab	0.862	42.668 a	12.763	7.263 ab	3.052	0.279 ab	0.126
RSFE 2	0.035 a	0.012	1.955 ab	2.305	35.596 a	12.660	6.785 ab	2.950	0.247 ab	0.119
STE 1	0.043 a	0.011	6.321 a	1.977	14.136 a	14.139	9.693 ab	4.052	0.133 ab	0.086
STE 2	0.038 a	0.011	5.327 ab	1.977	12.984 a	14.139	7.264 ab	3.508	0.107 ab	0.079

SAS Output for Crescent Glen.

Treatment	Ksat (cm/d)		NO_3^- ppm		TC ppm		TN ppm		TP ppm	
	Estimate	Std Err	Estimate	Std Err	Estimate	Std Err	Estimate	Std Err	Estimate	Std Err
Control			0.691 b	0.348	25.690 a	15.758	2.816 b	2.213	0.154 a	0.123
RSFE	0.042 a	0.007	0.336 b	0.151	29.813 a	15.440	2.803 b	2.144	0.067 a	0.047
STE	0.027 a	0.007	11.297 a	3.735	26.765 a	15.441	9.800 a	4.010	0.195 a	0.133
1 (30 cm)	0.052 a	0.009	1.674 a	0.456	27.914 a	15.496	4.705 a	2.772	0.131 a	0.073
2 (60 cm)			1.462 a	0.404	26.931 a	15.442	4.635 a	2.751	0.122 a	0.069
Control 1	0.080 a	0.023	0.739 b	0.416	24.646 a	16.289	2.657 b	2.209	0.153 a	0.127
Control 2			0.646 b	0.375	26.735 a	15.951	2.980 b	2.339	0.155 a	0.128
RSFE 1	0.047 ab	0.008	0.273 b	0.141	29.989 a	15.534	2.967 b	2.220	0.068 a	0.047
RSFE 2	0.038 ab	0.008	0.408 b	0.181	29.637 a	15.494	2.644 b	2.095	0.066 a	0.047
STE 1	0.030 ab	0.008	14.610 a	5.380	29.108 a	15.540	9.953 a	4.068	0.214 a	0.148
STE 2	0.024 b	0.009	8.725 a	3.140	24.422 a	15.495	9.648 a	4.006	0.177 a	0.123

Subsurface drip dispersal following lagoon treatment–A case for optimizing environmental protection

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ABSTRACT

The need to manage human waste has traditionally been driven by a desire to protect human health. Expectations have increased over time to achieve better protection, including the environment. Many older facilities have undergone a series of upgrades over time, most of which seek to achieve a higher level of protection. Bullards Beach State Park on the Pacific Coast of Oregon is a good example of this progression. The purpose of this presentation is to provide an overview of its wastewater treatment history and a bit more detail about the most recent upgrades.

Bullards Beach State Park consists of 198 campsites and was constructed in 1965. The initial wastewater treatment system consisted of an activated sludge package plant with spray irrigation on about 1.1 acres. A two-cell facultative lagoon system was installed in 1988 and the package plant was decommissioned. The existing spray irrigation site was used for the lagoon effluent. The park receives an average of 60 inches of precipitation per year. The lagoon system was not designed large enough to operate in a strict winter holding-summer irrigation pattern.

The requirements from the Oregon Department of Environmental Quality (DEQ) for spray irrigation changed after the lagoons were constructed to restrict irrigation to the growing season and limited to the water and nutrient needs of a crop. Subsurface dispersal is allowed year round and an innovative approach was developed to minimize the potential impact to human health and the environment while optimizing existing infrastructure and site features.

INTRODUCTION

Optimizing wastewater treatment requires a melding of science, policy, risk, and affordability. Innovative approaches often require policy elements, such as permits, to stray outside conventional boxes. The recently completed upgrades at Bullards Beach State Park represent a good example of an innovative approach that was considered and allowed, representing a cost-effective use of available resources to significantly reduce the risks to human health and the environment.

The existing facilities include three campground loops with a total of 104 full service recreational vehicle (RV) spaces, 81 limited service RV spaces (no individual sewer hook-ups), and 13 yurts, for a total of 198 campsites. The park is a popular destination on the Pacific coast of southern Oregon featuring an historic lighthouse, equestrian trails, beach access, frontage on the Coquille River, with nearby attractions including a world-class golf course at Bandon Dunes. The projected daily sewage flow for the previously mentioned facilities is 57,154 liters per day (L/d) or 15,100 gallons per day (gpd). Additional flows associated with camp staff, day use areas, and the park manager's residence could increase that number by about 10 percent to about 63,216 L/d (16,700 gpd). If a fourth camp loop were added in the future, consisting of 64 full service RV spaces, the projected daily sewage flow would be about 87,443 L/d (23,100 gpd), which was comparable to the limit listed in the existing permit when the project started (87,916 L/d or 23,225 gpd).

The original wastewater treatment system consisted of combination gravity collection system with strategic use of pumps to convey sewage and/or septic tank effluent to a single location. The combined sewage for the park was treated in an activated sludge package plant. The effluent was not disinfected. The resulting effluent was pumped over a dunal ridge to a spray irrigation site consisting of approximately 1.1 acres of scattered shore pine and an understory of various shrubs and miscellaneous grasses and forbs. Since there was no meaningful storage capacity within the system, irrigation was a daily occurrence regardless of weather conditions. The original system was set up before any meaningful guidance or rules were in effect pertaining to spray irrigation. Although the irrigation area was fenced, the buffer distances between the irrigation zones and the fence were less than what eventually became required. The irrigation site was on natural undulating (hummocky) terrain that was difficult to maintain. Work crews of prison labor were often brought in about once per year to trim limbs, remove brush, and cut the grass.

The package treatment system required a significant input of energy for aeration, and skilled operators, to achieve satisfactory results. In an effort to simplify operations and introduce a storage element, a 2-cell lagoon system was designed and installed in 1988. The lagoon system consists of a pair of facultative lagoons that operate in series. Each is about an acre in size and lined with a high density polyethylene (HDPE) membrane liner. The lagoons were plumbed together such that the water level would fluctuate simultaneously between a depth of 0.91 meters and 1.52 meters (3 feet and 5 feet - normal operating depths). The total depth is 2.44 meters (8 feet), which allows for 0.91 meters (3 feet) of freeboard. Implementation of new rules governing irrigation with reclaimed water a few years later required effluent to be applied as a beneficial use, meaning as a source of water and nutrients during the growing season. The lagoons were not designed to provide adequate storage to avoid irrigation during the wet season. The area receives an average of about 152 centimeters (60 inches) of precipitation annually; most of which occurs in the late fall, winter, and early spring. Therefore it was not possible to avoid irrigation during the non-growing season. Typical irrigation volumes applied to sandy soils during the rainy season were suspected of contributing potential pollutants to groundwater.

The Oregon Parks and Recreation Department (OPRD) faced either upgrading the level of treatment (to include disinfection) prior to irrigation or develop an acceptable alternative. Conventional subsurface dispersal was proposed for consideration in 2008 but that proposal apparently did not include provisions for enhanced treatment. In 2009, OPRD commissioned a study to evaluate the feasibility of using constructed wetlands followed by rapid infiltration as a means of providing an alternative to the spray irrigation system.

A request for proposals to complete the design of this alternative was solicited. In reviewing the conceptual design, Cascade Earth Sciences (CES) was not confident in several of the treatment assumptions and was concerned that the DEQ would not accept rapid infiltration as being adequately protective of groundwater quality. Based on the CES analysis, it was decided that a different alternative would be proposed for consideration by OPRD and DEQ. This alternative consisted of fine filtration and distribution across a large area using subsurface drip irrigation.

MATERIALS AND METHODS

A stated goal in the previous assessment of alternatives was to have effluent nitrogen concentrations below 5 milligrams per liter (mg/L). Although the DEQ rules for managing recycled water seek to limit irrigation to the growing season, the rules for subsurface dispersal are based on distribution to the soil year round.

Pond Balances. A summary of climate data for the nearby Bandon weather station is presented in Table 1. A detailed water balance analysis was conducted to verify an assumption that the existing collection and treatment system components were acceptably watertight. Table 2 summarizes the modeled inputs, outputs, and lagoon depths for the period between November 1, 2008 and October 31, 2009. The difference between the modeled lagoon depths and actual lagoon depths was about 16.5 centimeters (6.5 inches) in the lagoons (ending depth of 91 cm (36 inches) in the model versus 74.9 cm (29.5 inches) measured in the field). This represents an average daily difference of less than 3,785 L (1,000 gallons). As a result, infiltration/inflow (I/I) did not appear to be a significant issue.

Table 1. Temperature, precipitation, and evaporation
Bullards Beach State Park - Coos County, Oregon

Month	Average daily maximum	Average daily minimum	Average daily	Average number of days with lows of 32°F degrees or less	Average precipitation	Average snowfall	Average pan evaporation	Estimated free water (lake) evaporation ‡
	degrees Fahrenheit †			days	inches			
January	53.5	38.5	46.0	6.5	10.09	0.6	1.0	0.9
February	55.2	39.3	47.3	4.8	7.47	0.1	1.2	1.0
March	55.6	39.8	47.7	3.2	7.28	0	1.8	1.5
April	57.5	41.2	49.4	1.2	4.46	0	2.5	2.2
May	60.7	44.5	52.6	0.3	3.07	0	3.7	3.2
June	63.9	48.4	56.2	0.0	1.48	0	4.1	3.5
July	66.1	50.6	58.4	0.0	0.38	0	4.8	4.1
August	67.1	50.5	58.8	0.0	0.75	0	4.0	3.4
September	66.8	48.0	57.4	0.0	1.58	0	2.8	2.4
October	63.1	44.5	53.8	0.5	4.47	0	1.6	1.4
November	57.6	41.8	49.7	2.7	8.46	0	1.0	0.9
December	53.9	39.1	46.5	5.4	9.92	0.1	1.0	0.9
Yearly								
Average	**60.1**	**43.9**	**52.0**					
Total				**24.6**	**59.41**	**0.8**	**29.50**	**25.37**

† Data based on records at Bandon 2 NNE Weather Station. Period of record, 1948 through 2006 (Western Regional Climate Center website url: http://www.wrcc.dri.edu).

‡ Based on average pan evaporation data for Astoria AP, adjusted with a free water surface evaporation coefficient of 0.86 (NOAA Technical Report NWS 33).

Next, several additional water balance scenarios were analyzed to determine likely operating conditions for a variety of circumstances, such as current conditions, record-level high precipitation, potential expansion, etc. Using the 2008-2009 conditions previously described and a daily discharge of 38,800 L (10,250 gallons), the lagoon levels could have been maintained between the minimum and maximum operating depths of 91 cm (36 inches) and 152 cm (60 inches), respectively (Table 3). Changing the precipitation to average values increased the daily discharge to 48,718 L (12,870 gallons) (Table 4). Increasing the 2008-2009 actual flows by 30 percent to account for potential expansion, coupled with high rainfall using data from the wettest year in over 50 years (November 1996 through October 1997) increased the daily discharge to 68,137 L (18,000 gallons) (Table 5). A worst-case scenario assuming peak projected flows of 87,916 L (23,225 gallons) every day, coupled with high rainfall as described in the previous scenario, resulted in an annual average daily discharge flow of 126,811 L (33,500 gallons) (149,924 L/d or 39,500 gpd for November through February and October, and 113,562 L/d or 30,000 gpd for March through September) (Table 6).

Table 2. Pond balance (2008-2009 rainfall) as irrigated
Bullards Beach State Park - Coos County, Oregon

Month	Beginning balance		Inputs: Flow†		Inputs: Precipitation‡		Outputs: Evaporation§		Outputs: Seepage¶		Outputs: Irrigation#		Outputs: Discharge		Ending balance	
	inches	MG	inches	MG	inches	MG	inches	MG	inches	MG	inches	MG	inches	MG	inches	MG
November	34	1.86	1.00	0.055	6.10	0.368	0.86	0.037	0.00	0	0.00	0.00	0.00	0.00	41	2.25
December	41	2.25	0.88	0.048	9.11	0.549	0.86	0.037	0.00	0	0.00	0.00	0.00	0.00	51	2.81
January	51	2.81	1.49	0.082	3.54	0.213	0.86	0.037	0.00	0	0.00	0.00	0.00	0.00	56	3.07
February	56	3.07	2.01	0.110	7.17	0.432	1.03	0.044	0.00	0	2.34	0.13	0.00	0.00	63	3.44
March	63	3.44	2.99	0.164	7.65	0.461	1.55	0.066	0.00	0	10.58	0.58	0.00	0.00	62	3.42
April	62	3.42	4.34	0.238	2.87	0.173	2.15	0.092	0.00	0	10.89	0.60	0.00	0.00	57	3.14
May	57	3.14	6.75	0.370	2.33	0.140	3.18	0.136	0.00	0	0.00	0.00	0.00	0.00	64	3.52
June	64	3.52	6.95	0.381	0.49	0.030	3.53	0.150	0.00	0	3.02	0.17	0.00	0.00	66	3.61
July	66	3.61	6.07	0.333	0.10	0.006	4.13	0.176	0.00	0	6.11	0.34	0.00	0.00	63	3.44
August	63	3.44	3.04	0.167	0.45	0.027	3.44	0.147	0.00	0	3.28	0.18	0.00	0.00	60	3.31
September	60	3.31	1.29	0.071	1.09	0.066	2.41	0.103	0.00	0	11.41	0.63	0.00	0.00	49	2.71
October	49	2.71	1.37	0.075	3.48	0.210	1.38	0.059	0.00	0	17.18	0.94	0.00	0.00	36	2.00
Total			**38.2**	**2.1**	**44.4**	**2.7**	**25.4**	**1.1**	**0.00**	**0**	**64.8**	**3.6**	**0.0**	**0.0**		

NOTES:

Abbreviation: MG = million gallons.

† Based on reported flows for November 2008 through October 2009.

‡ Based on the measured precipitation at the AgriMet weather station at Bandon and the total area both lagoons.

§ Based on the lake evaporation rates shown in Table 1 applied across the surface area of both lagoons.

¶ Based on an assumed seepage rate of 0.00 inches per day for the existing lagoons since they are lined with a heavy duty HDPE liner.

Based on reported irrigation volumes for November 2008 through October 2009.

Table 3. Pond balance (2008-2009 rainfall) as proposed
Bullards Beach State Park - Coos County, Oregon

Month	Beginning balance		Inputs: Flow†		Inputs: Precipitation‡		Outputs: Evaporation§		Outputs: Seepage¶		Outputs: Irrigation		Outputs: Discharge#		EndingbBalance	
	inches	MG	inches	MG	inches	MG	inches	MG	inches	MG	inches	MG	inches	MG	inches	MG
November	36	1.97	1.00	0.055	6.10	0.368	0.86	0.037	0.00	0	0.00	0	5.61	0.31	37	2.05
December	37	2.05	0.88	0.048	9.11	0.549	0.86	0.037	0.00	0	0.00	0	5.61	0.31	42	2.31
January	42	2.31	1.49	0.082	3.54	0.213	0.86	0.037	0.00	0	0.00	0	5.61	0.31	41	2.26
February	41	2.26	2.01	0.110	7.17	0.432	1.03	0.044	0.00	0	0.00	0	5.61	0.31	45	2.45
March	45	2.45	2.99	0.164	7.65	0.461	1.55	0.066	0.00	0	0.00	0	5.61	0.31	49	2.70
April	49	2.70	4.34	0.238	2.87	0.173	2.15	0.092	0.00	0	0.00	0	5.61	0.31	49	2.71
May	49	2.71	6.75	0.370	2.33	0.140	3.18	0.136	0.00	0	0.00	0	5.61	0.31	51	2.78
June	51	2.78	6.95	0.381	0.49	0.030	3.53	0.150	0.00	0	0.00	0	5.61	0.31	50	2.73
July	50	2.73	6.07	0.333	0.10	0.006	4.13	0.176	0.00	0	0.00	0	5.61	0.31	47	2.59
August	47	2.59	3.04	0.167	0.45	0.027	3.44	0.147	0.00	0	0.00	0	5.61	0.31	42	2.33
September	42	2.33	1.29	0.071	1.09	0.066	2.41	0.103	0.00	0	0.00	0	5.61	0.31	37	2.05
October	37	2.05	1.37	0.075	3.48	0.210	1.38	0.059	0.00	0	0.00	0	5.61	0.31	36	1.97
Total			**38.2**	**2.1**	**44.4**	**2.7**	**25.4**	**1.1**	**0.00**	**0**	**0.0**	**0.0**	**67.3**	**3.7**		

NOTES:

Abbreviation: MG = million gallons.

† Based on reported flows for November 2008 through October 2009.

‡ Based on the measured precipitation at the AgriMet weather station at Bandon and the total area both lagoons.

§ Based on the lake evaporation rates shown in Table 1 applied across the surface area of both lagoons.

¶ Based on an assumed seepage rate of 0.00 inches per day for the existing lagoons since they are lined with a heavy duty HDPE liner.

Based on an average discharge volume of 10,250 gallons per day throughout the year.

Table 4. Pond balance (2008-2009 flows, average rainfall)
Bullards Beach State Park - Coos County, Oregon

Month	Beginning balance		Inputs				Outputs								Ending balance	
			Flow†		Precipitation‡		Evaporation§		Seepage¶		Irrigation		Discharge#			
	inches	MG	inches	MG	inches	MG	inches	MG	inches	MG	inches	MG	inches	MG	inches	MG
November	36	1.97	1.00	0.055	8.46	0.510	0.86	0.037	0.00	0	0.00	0	6.99	0.38	39	2.12
December	39	2.12	0.88	0.048	9.92	0.598	0.86	0.037	0.00	0	0.00	0	6.99	0.38	43	2.35
January	43	2.35	1.49	0.082	10.09	0.608	0.86	0.037	0.00	0	0.00	0	6.99	0.38	48	2.62
February	48	2.62	2.01	0.110	7.47	0.450	1.03	0.044	0.00	0	0.00	0	6.99	0.38	50	2.75
March	50	2.75	2.99	0.164	7.28	0.439	1.55	0.066	0.00	0	0.00	0	6.99	0.38	53	2.90
April	53	2.90	4.34	0.238	4.46	0.269	2.15	0.092	0.00	0	0.00	0	6.99	0.38	53	2.93
May	53	2.93	6.75	0.370	3.07	0.185	3.18	0.136	0.00	0	0.00	0	6.99	0.38	54	2.97
June	54	2.97	6.95	0.381	1.48	0.089	3.53	0.150	0.00	0	0.00	0	6.99	0.38	53	2.91
July	53	2.91	6.07	0.333	0.38	0.023	4.13	0.176	0.00	0	0.00	0	6.99	0.38	49	2.70
August	49	2.70	3.04	0.167	0.75	0.045	3.44	0.147	0.00	0	0.00	0	6.99	0.38	43	2.39
September	43	2.39	1.29	0.071	1.58	0.095	2.41	0.103	0.00	0	0.00	0	6.99	0.38	38	2.07
October	38	2.07	1.37	0.075	4.47	0.269	1.38	0.059	0.00	0	0.00	0	6.99	0.38	36	1.97
Total			**38.2**	**2.1**	**59.4**	**3.6**	**25.4**	**1.1**	**0.00**	**0**	**0.00**	**0**	**83.9**	**4.6**		

NOTES:

Abbreviation: MG = million gallons.

† Based on reported flows from 2009.

‡ Based on the wettest year in over 50 years (November 1996 through October 1997) at the Bandon 2 NNE weather station and the total area both lagoons.

§ Based on the lake evaporation rates shown in Table 1 applied across the surface area of both lagoons.

¶ Based on an assumed seepage rate of 0.00 inches per day for the existing lagoons since they are lined with a heavy duty HDPE liner.

Based on an average discharge volume of 12,870 gallons per day throughout the year.

Table 5. Pond balance (expanded flows, high rainfall)
Bullards Beach State Park - Coos County, Oregon

Month	Beginning balance		Inputs				Outputs								Ending balance	
			Flow†		Precipitation‡		Evaporation§		Seepage¶		Irrigation		Discharge#			
	inches	MG	inches	MG	inches	MG	inches	MG	inches	MG	inches	MG	inches	MG	inches	MG
November	36	1.97	1.30	0.072	16.26	0.980	0.86	0.037	0.00	0	0.00	0	9.84	0.540	45	2.45
December	45	2.45	1.14	0.062	19.25	1.160	0.86	0.037	0.00	0	0.00	0	9.84	0.540	56	3.10
January	56	3.10	1.94	0.107	11.41	0.688	0.86	0.037	0.00	0	0.00	0	9.84	0.540	60	3.31
February	60	3.31	2.61	0.143	2.46	0.148	1.03	0.044	0.00	0	0.00	0	9.84	0.540	55	3.02
March	55	3.02	3.89	0.213	9.21	0.555	1.55	0.066	0.00	0	0.00	0	9.84	0.540	58	3.18
April	58	3.18	5.64	0.309	3.52	0.212	2.15	0.092	0.00	0	0.00	0	9.84	0.540	56	3.07
May	56	3.07	8.77	0.481	2.52	0.152	3.18	0.136	0.00	0	0.00	0	9.84	0.540	55	3.03
June	55	3.03	9.03	0.495	1.43	0.086	3.53	0.150	0.00	0	0.00	0	9.84	0.540	53	2.92
July	53	2.92	7.89	0.433	0.50	0.030	4.13	0.176	0.00	0	0.00	0	9.84	0.540	49	2.67
August	49	2.67	3.96	0.217	1.32	0.080	3.44	0.147	0.00	0	0.00	0	9.84	0.540	42	2.28
September	42	2.28	1.68	0.092	5.38	0.324	2.41	0.103	0.00	0	0.00	0	9.84	0.540	37	2.05
October	37	2.05	1.78	0.098	6.70	0.404	1.38	0.059	0.00	0	0.00	0	9.84	0.540	36	1.96
Total			**49.6**	**2.7**	**80.0**	**4.8**	**25.4**	**1.1**	**0.0**	**0.0**	**0.0**	**0.0**	**118.1**	**6.5**		

NOTES:

Abbreviation: MG = million gallons.

† Based on reported flows from 2009 increased 30 percent to account for the potential for a future 4th camp loop of 64 additional full services spaces.

‡ Based on the wettest year in over 50 years (November 1996 through October 1997) at the Bandon 2 NNE weather station and the total area both lagoons.

§ Based on the lake evaporation rates shown in Table 1 applied across the surface area of both lagoons.

¶ Based on an assumed seepage rate of 0.00 inches per day for the existing lagoons since they are lined with a heavy duty HDPE liner.

Based on an average discharge volume of 18,000 gallons per day throughout the year.

Table 6. Pond balance (permit flows, high rainfall)
Bullards Beach State Park - Coos County, Oregon

Month	Beginning balance		Inputs				Outputs								Ending balance	
			Flow†		Precipitation‡		Evaporation§		Seepage¶		Irrigation		Discharge#			
	inches	MG	inches	MG	inches	MG	inches	MG	inches	MG	inches	MG	inches	MG	inches	MG
November	36	1.97	12.70	0.697	16.26	0.980	0.86	0.037	0.00	0	0.00	0	21.60	1.19	44	2.43
December	44	2.43	13.13	0.720	19.25	1.160	0.86	0.037	0.00	0	0.00	0	21.60	1.19	56	3.09
January	56	3.09	13.13	0.720	11.41	0.688	0.86	0.037	0.00	0	0.00	0	21.60	1.19	60	3.27
February	60	3.27	11.86	0.650	2.46	0.148	1.03	0.044	0.00	0	0.00	0	21.60	1.19	52	2.84
March	52	2.84	13.13	0.720	9.21	0.555	1.55	0.066	0.00	0	0.00	0	16.41	0.90	57	3.15
April	57	3.15	12.70	0.697	3.52	0.212	2.15	0.092	0.00	0	0.00	0	16.41	0.90	56	3.07
May	56	3.07	13.13	0.720	2.52	0.152	3.18	0.136	0.00	0	0.00	0	16.41	0.90	53	2.91
June	53	2.91	12.70	0.697	1.43	0.086	3.53	0.150	0.00	0	0.00	0	16.41	0.90	48	2.64
July	48	2.64	13.13	0.720	0.50	0.030	4.13	0.176	0.00	0	0.00	0	16.41	0.90	42	2.31
August	42	2.31	13.13	0.720	1.32	0.080	3.44	0.147	0.00	0	0.00	0	16.41	0.90	38	2.07
September	38	2.07	12.70	0.697	5.38	0.324	2.41	0.103	0.00	0	0.00	0	16.41	0.90	38	2.09
October	38	2.09	13.13	0.720	6.70	0.404	1.38	0.059	0.00	0	0.00	0	21.60	1.19	36	1.97
Total			**154.5**	**8.5**	**80.0**	**4.8**	**25.4**	**1.1**	**0.00**	**0**	**0.00**	**0**	**222.9**	**12.2**		

NOTES:

Abbreviation: MG = million gallons.

† Based on the listed design flow of 23,225 gallons per day.

‡ Based on the wettest year in over 50 years (November 1996 through October 1997) at the Bandon 2 NNE weather station and the total area both lagoons.

§ Based on the lake evaporation rates shown in Table 1 applied across the surface area of both lagoons.

¶ Based on an assumed seepage rate of 0.00 inches per day for the existing lagoons since they are lined with a heavy duty HDPE liner.

Based on an average discharge volume of 39,500 gallons per day for November through February, 30,000 gpd for March through September, 39,500 gallons per day for October, and an annual average of 33,500 gallons per day.

Soil Conditions. The lagoons were constructed on a stabilized and gently sloping dunal surface about 6.1 to 9.1 meters (20 to 30 feet) above mean sea level (MSL) with a relatively shallow water table. East of the lagoons was an undulating (hummocky) dunal ridge that rose as high as about 24.4 meters (80 feet) above MSL that was being considered for the subsurface drip field. The soils are mapped in the published soil survey as Dune Land (USDA, 1989). The soils were initially evaluated with an extension auger to a depth of as much as 3.5 meters (11.5 feet) at a location estimated to be about 15.2 meters (50 feet) above MSL. Relatively uniform medium to fine sand was observed throughout the depth evaluated. A nearby low spot, estimated to be about 9.1 meters (30 feet) above MSL, was evaluated and similar conditions were observed to a depth of about 1.2 meters (4 feet) where saturated conditions occurred. This data correlated relatively well with open water in an area to the west used as a borrow area during lagoon construction and the static water level measured in the water supply well, located about 396 meters (1,300 feet) to the east, when it was drilled in October 1992.

Several backhoe test holes were subsequently evaluated to confirm conditions throughout the proposed subsurface drip field site. Archaeological observation was required during excavation due to the confirmed presence of Native American artifacts in the area. No artifacts were identified during either the evaluation phase or any subsequent construction activities.

Sizing Based on Hydraulics. The sizing of the subsurface drip field was evaluated on the basis of both potential peak flow conditions and nitrogen loading. A review of the hydraulic design guidance provided by the two primary manufacturers of subsurface drip components for wastewater indicate sizing for septic tank effluent in unstructured sands at 12.3 liters per square meter per day ($L/m^2/d$) or 0.3 gallons per square foot per day ($g/ft^2/d$) and treated effluent at 32.7

to 49 L/m^2/d (0.8 to 1.2 g/ft^2/d). The minimum total area for a drip field would be about 1.21 hectares (3.0 acres) for septic tank effluent and 0.45 hectares (1.1 acres) for treated effluent, based on a maximum monthly average loading rate of 149,524 L/d (39,500 gpd).

Sizing Based on Nutrient Loading. The flow-weighted total nitrogen value in the lagoon effluent ranged from 320 kilograms (kg) or 705 pounds in 2008 to 402 kg (887 pounds) in 2009, for a 2-year average of about 361 kg (796 pounds) per year. Increasing the total nitrogen by roughly 30 percent to account for potential future loading from a fourth camp loop and adjusting for normal precipitation, the average total nitrogen concentration in the lagoon effluent is expected to be about 23.6 mg/L. Using the average relative proportions from the 2008 – 2009 data, an average of about 16.4 mg/L is expected to be in the organic-N form and about 7.2 mg/L is expected to be in the nitrate-N and ammonium-N forms. Potential N mechanisms include - the organic nitrogen will likely be mineralized over time to available forms, uptake will occur from the cover crop (grass), immobilization will likely aid in the formation of soil organic matter, and some losses will likely occur as a result of denitrification and deep percolation. These processes will reach equilibrium over the course of consistent long term management. Losses of nitrogen to denitrification, volatilization, and soil storage from primary or secondary treated effluent can range from 15 to 50 percent of the total applied (EPA, 2006), with 25 percent assumed for the purpose of this analysis.

One distinct advantage of the drip system is the delivery of water and nutrients evenly across the entire application area on a 61 cm by 61 cm (2-foot by 2-foot) grid and within the active root zone (an average depth of about 25.4 cm (10 inches) below the surface). If we further assume that an established stand of grasses will take up 100 pounds of nitrogen per acre (lb N/ac) per year and retain an additional 55 lb N/ac in the development of soil organic matter, then the long term net nitrogen concentration in percolate losses below the root zone will be less than 5 mg/L so long as the active drip field is at least 1.17 hectares (2.9 acres) in size. This also corresponds closely to the size based on the potential peak month hydraulic loading.

Design. Based on the irregular topography of the dunal ridge, it was decided that grading the unstructured sand would create a more uniform shape in order to facilitate easier installation and maintenance. Since the sand was structureless (single grain) with little or no horizon development, this was viewed as a relatively low risk approach as compared to most other soils. The existing vegetation was primarily shore pine and invasive (non-native) beach grass. Approximately 1.78 hectares (4.4 acres) were cleared with the trees being chipped. The sand was cut from the high spots and filled in the low spots with the final surface being relatively level from east to west and gently sloping at between 4 and 5 percent from south to north over a total area of about 1.21 hectares (3.0 acres) with the remainder being transition slopes around the edges. This surface is safe to maintain with a riding lawn mower.

A process flow schematic is shown in Figure 1. One goal was to use as much of the existing infrastructure as possible. No modifications were made to the previously described collection system components other than adding a magnetic flow meter to the pipe just prior to the first lagoon in order measure influent flows. The existing intake from second lagoon to the irrigation pumps was an open pipe and not screened. Rough-skinned newts, an amphibian salamander-like creature, thrived in the lagoon environment and were a constant source of problems with the pumps. Installing a screen within the lagoon was considered but would have required draining the lagoon and modifying the existing liner system. The concept was rejected in favor of an alternative that could be spliced into the pipe outside the lagoon (between the intake and the pumps). A 2-compartment tank was installed that included a pair of 1/8-inch mesh screens to prevent larger solids from passing from the first compartment to the second. Newts are persistent and modifications were necessary to block their passage.

A new skid-mounted duplex pump assembly (7.5 Hp Berkley with 230 VAC single phase motors) and integrated disk filter headworks (2-inch Arkal 100 micron) was designed in cooperation with Jim Prochaska, P.E., and built by JNM Technologies (Bryan, Texas) that included automated backflushing of the disk filters based on a pre-determined pressure-differential. JNM Technologies also custom-built an assembly for the zone valves and field flush valve. Netafim drip tubing with pressure-compensating emitters rated at 2.27 L/hr (0.6 g/hr) with 51 cm (24-inch) spacing was installed on 51 cm1 (24-inch) centers in the drip field. A total of eight zones, each consisting of 20 laterals 122 meters (400 feet) long were installed (overall dimensions of 97.5 meters by 122 meters (320 feet by 400 feet), or about 1.17 hectares (2.9 acres)). Based on the unique circumstances at this site, one pump is active at a time and doses two zones at a time during normal operations and one zone at a time during field flushing. Each time the pump comes on, all eight zones get dosed as four pairs in consecutive series. The daily flow to the drip field is determined by the off time that varies according to the liquid level.

The system is controlled by an Orenco® custom TeleComm™ (TCOM) telemetry capable control panel, which provides a high level of user interface capability and detailed data storage. A telephone line was not readily available, so the panel was equipped with a cellular-based communication interface that enabled remote access for system monitoring and management as well as remote alarm notification. This proved invaluable, particularly during system start-up when newts were disrupting operations and technical support was at least three hours away.

RESULTS AND DISCUSSION

The system was completed and operational in May 2012. Prior to the recent project, the lagoon system was pumped down to an operating depth as low as 30 inches in the fall to maximize storage capacity going into the wet season. As stated previously, the design assumed the original operating conditions would not change and it was assumed that the operating levels would fluctuate between a depth of 91 cm (36 inches) and 152 cm (60 inches). This enables a steadier discharge rate throughout the year by storing rainfall during the wet season and metering it out during the dry season. However, the local operator indicated a preference to maintain the lagoons closer to a steady operating level (near the 152 cm (60-inch) maximum depth). This results in increased seasonal discharge flow variation (higher discharge rates during the wetter months and lower discharge rates during the drier months). Even with this change, the there is still at least one discharge cycle per day to help maintain the grass cover crop on the drip field during the summer and as many as 12 cycles per day during the wettest winter periods.

The first full year of monitoring that is comparable to what was modeled prior to design is represented by data from November 2012 through October 2013. The system is operating within the range of pre-design assumptions. A magnetic flow meter was installed on the influent piping ahead of the first lagoon as part of the project which has increased the confidence in the incoming flow and confirmed that infiltration is not a significant issue. Average daily influent flows ranged from as low as 15,142 L/d (4,000 gpd) in November 2012 to as high as 37,097 L/d (9,800 gpd) in July 2013 for an annual average of 21,955 L/d (5,800 gpd). Average daily effluent flows ranged from a low of about 16,277 L/d (4,300 gpd) in July 2012 to a high of 86,686 L/d (22,900 gpd) in December 2012 for an annual average of 40,882 L/d (10,800 gpd). These values compare favorably with the annual averages for a comparable period evaluated for the design (November 2008 through October 2009 - influent average of 21,577 L/d (5,700 gpd) and effluent average of 36,718 L/d (9,700 gpd)). The higher effluent annual flow in the 2012-13 time period is primarily attributable to higher rainfall (113 cm (44.4 inches) in 2008-09 compared to 141 cm (55.4 inches) in 2012-13).

The total annual effluent loading for the 2012-13 period over the 1.2 hectare (3-acre) drip field was approximately 123 cm (48.6 inches), or an average of 0.33 cm (0.13 inches) per day. Accounting for a conservative estimate of evapotranspiration at 56 cm (22 inches) for lawns (based on data from the nearby AgriMet weather station located approximately 6.84 km (4.25 miles) south of the site), approximately 67.6 cm (26.6 inches) percolated beyond the root zone U.S. Bureau of Reclamation, 2013). Assuming all 141 cm (55.4 inches) of precipitation are 100 percent effective, and additive to the percolate, results in a total of 208 cm (82 inches) moving below the root zone toward groundwater in the 2012-13 period.

The current permit requires quarterly effluent sampling. Parameters include five-day biochemical oxygen demand (BOD_5), total suspended solids (TSS), total Kjeldahl nitrogen (TKN), and nitrate-nitrogen (NO_3-N). The prior permit required effluent sampling on a monthly basis when irrigating. Eight samples were collected in 2008-09 period described previously. BOD_5 ranged from 13 to 64 mg/L, with an average of 40 mg/L; whereas four samples for 2012-13 ranged from 12 to less than 105 mg/L, with an average of 30 mg/L. TSS during the former period ranged from 62 to 187.5 mg/L, with an average of 103 mg/L, compared to 6 to 176 mg/L with an average of 68 mg/L for the current period. Total nitrogen (TKN + NO_3-N) averaged 26.5 mg/L with a range of 9.9 to 67.9 mg/L in the former period, compared to an average of 17.5 mg/L with a range of 11.8 to 24.3 mg/L in the current period.

CONCLUSIONS

Removal of particles larger than 100 microns by the disk filters is primarily responsible for the reduction in the average concentrations. The lower nitrogen concentrations resulted in total nitrogen loading of 278 kg (613 pounds) in the current period compared to 402 kg (887 pounds) in the former period. The pattern of the grass in the sandy soil clearly matches the pattern of the emitters with coverage in between gradually filling in. Even accounting for reduced coverage (estimated at 50%) and reducing the nitrogen uptake potential accordingly, the estimated annual flow-weighted average concentration of NO_3-N in percolate below the root zone for the 2012-13 period (using the same assumptions as in the pre-design analysis) would be less than 3 mg/L. Subsurface drip irrigation appears to represent a reasonable approach to provide an environmentally sound and cost effective alternative for managing facultative lagoon effluent.

Acknowledgements. Many people contributed to the successful formulation and design of this project. Special thanks are offered to Ben Fisher, Park Manager; Chuck Littlejohn, Park Ranger and Wastewater System Operator; as well as John Seevers, Project Manager for OPRD; Jim Prochaska, PE with JNM Technologies for working with me through the unique requirements and circumstances of this site; Carey Bailey with Orenco Systems, Inc. for all his effort designing the wireless TCOM panel; and many of my co-workers including Tim Otis, PE, Project Engineer, for his many contributions throughout the project.

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Filtration of Stormwater Contaminants in Bioretention Cells

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ABSTRACT

Heavy metals like copper and zinc are ubiquitous in stormwater runoff, and stormwater is often introduced into surface waters without treatment. Thus, receiving waters are impacted, with serious consequences for aquatic organisms and the food web. Bioretention systems are useful for reducing the contaminant load of stormwater and managing the amount of stormwater introduced to receiving waters. However, the most effective compositions of bioretention systems for pollutant removal need to be determined. We built 16 mesocosms with different porous media to study contaminant retention capacities. We used four media (mix15: 60% sand, 15% compost, 15% shredded cedar bark, 10% water treatment residuals; mix20: 80% sand, 20% compost; mix30: 60% sand, 30% compost, 10% water treatment residuals; mix40: 60% sand, 40% compost), with each treatment replicated four times. Mesocosms have been continuously treating ambient stormwater runoff since 2011. During 2013, we spiked stormwater runoff from two events with copper and zinc, and monitored removal in the four mesocosm treatments. On average, the mesocosms reduced the total load of heavy metals significantly (copper: 74 and 65%; zinc: 98 and 97%; for the first and second storm, respectively), irrespective of the treatment media.

INTRODUCTION

Land use development and associated stormwater are primary causes of fresh and marine water degradation. Increased runoff volume, peak flows and flow durations accelerate sediment delivery, scour stream channels, reduce habitat complexity, and change hydroperiods in wetlands. A wide range of pollutants are associated with stormwater flows, including heavy metals, oil and grease, pesticides, polycyclic aromatic hydrocarbons, sediment, and nutrients (nitrogen and phosphorus). In some land use settings, pollutant concentrations in stormwater runoff can exceed levels that are considered acutely toxic. Pollutant concentrations can also exceed chronic toxicity levels in urbanized streams (Kayhanian et al. 2008).

The current structural approach to stormwater has limitations for fully mitigating the flow from and water quality impacts of urban development. Increasingly, stormwater engineers and designers are exploring and implementing distributed, low-impact development (LID) strategies that seek to preserve the natural hydrologic regime of a watershed by managing stormwater as close to its source as possible.

Research focused on LID practices has increased recently in the U.S. Washington State University and project partners constructed the first university LID Research Program in the western U.S. One of the focuses of the program is bioretention: full-scale replicated research plots can be used to test the water quality treatment and flow control performance of these systems. The objective of the mesocosm research presented here is to examine the hydrologic and water quality treatment performance of various bioretention soil mixes.

MATERIALS AND METHODS

We constructed a stormwater research facility that collects stormwater from impervious surfaces, including roof tops and pavement, with an area of 6670 m^2. Runoff from approximately 25% of this area (1674 m^2) is routed to a high-density polyethylene (HDPE) cistern (11,370 L) to store and deliver stormwater to16 mesocosms. The mesocosms contain four replicates of four different bioretention soil mixtures (Table 1).

Each mesocosm was constructed with a 152-cm-diameter by 132-cm-deep HDPE media tank to hold the bioretention soil mix (Fig. 1). The bottom of each media tank was filled with coarse sand to a depth of 30 cm as a drainage layer, followed by 61 cm of the corresponding bioretention soil mix. Stormwater enters the tanks through a manifold constructed of PVC plastic piping (5-cm-diameter) perforated with drilled holes that distribute water across the surface of the soil. A slotted PVC underdrain pipe (2.5-cm-diameter) within the aggregate layer drains the media tank. Effluent flow rate was determined with tipping buckets. Samples for water quality analysis were from flow-weighted composite samples across a storm event.

During a rain storm the surface runoff from the collection area was directed into the cistern. The runoff water at this research facility is relatively clean and contains very few contaminants. To mimic surface runoff from different developments – like industrial, commercial or residential areas – we can dose the stormwater in the cistern before it is delivered to the mesocosms. Table 2 shows the dosing of copper and zinc used in this study, representing a medium commercial or industrial development. A control outlet from the cistern bypasses the mesocosms and flows directly to a water quality sample station and tipping bucket unit. Performance of the mesocosms was assessed by comparing the mesocosm outflow concentrations and loads to concentrations and loads from the control outlet.

Pollutant reduction efficiency for each bioretention soil mix was assessed by computing the pollutant concentration reduction and the pollutant load reduction (Ecology 2008). The reduction (in percent) in pollutant concentration during each individual storm (ΔC) was calculated as:

$$\Delta C = 100 \times \frac{(C_{\mathrm{in}} - C_{\mathrm{eff}})}{C_{\mathrm{in}}}$$

Where:

ΔC	=	pollutant concentration reduction in percent
C_{in}	=	flow-weighted influent pollutant concentration
C_{eff}	=	flow-weighted effluent pollutant concentration

The pollutant load reduction (in percent) for individual storms (ΔL) was calculated as:

$$\Delta L = 100 \times \frac{\left((C_{\mathrm{in}} \times V_{\mathrm{i,in}}) - (C_{\mathrm{eff}} \times V_{\mathrm{i,eff}})\right)}{(C_{\mathrm{in}} \times V_{\mathrm{i,in}})}$$

Where:

ΔL	=	pollution load reduction in percent
C_{in}	=	flow-weighted influent pollutant concentration

$V_{i,in}$ = inflow volume of storm i

$V_{i,eff}$ = effluent volume of storm i

C_{eff} = flow-weighted effluent pollutant concentration

To assess the performance of the different bioretention mixes we performed a Friedman test (Conover, 1980) on the pollutant concentration reduction and on the pollutant load reduction.

RESULTS AND DISCUSSION

We monitored two storm events in 2013. Storm A, on April 10, had 5.8 mm precipitation depth over 2.5 h. Storm B, on May 27, had 14.7 mm precipitation depth over 12 h. The concentrations and loads of total and dissolved copper and zinc for the four different soil mixes are reported in Table 3. Overall, the concentrations and loads of copper and zinc were reduced significantly for all mesocosm treatments compared to the control outlet. Total and dissolved copper concentration and load reduction was higher for storm A than storm B. Concentration reduction for total copper was 73.8% and 65.2%, and load reduction was 74.3% and 65.4% for storm A and B, respectively. Concentration reduction for dissolved copper was 68.8% and 34.8%, and load reduction was 69.4% and 35.2% for storm A and B, respectively. Zinc was removed at very high percentages, with no difference between the storms. Concentration reduction for total zinc was 97.7% and 97.1%, and load reduction was 97.8% and 97.1% for storm A and B, respectively. Concentration reduction for dissolved zinc was 97.1% and 95.9%, and load reduction was 97.1% and 95.9% for storm A and B, respectively. We found no significant differences in the performance of the different bioretention mixes for either copper or zinc removal ($\alpha = 0.05$).

Removal of dissolved copper was more than 30% and the removal of dissolved zinc was more than 60% for both storm events. Thus these bioretention cells meet the requirements for an enhanced treatment facility in the state of Washington (Ecology 2008).

Although highly effective at removing copper and zinc from stormwater runoff, none of the four soil mixes performed significantly better than the others. A mix of 60% sand and 40% compost removes heavy metals as efficiently as mixtures with more than two components. Differences in removal rates among soil treatments may still occur over period of operation longer than two years.

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Table 1. Bioretention soil mixes. All percentages are per volume.

Mix	Sand (%)	Compost (%)	WTR† (%)	Shredded Cedar Bark (%)
Mix15	60	15	15	10
Mix20	80	20	0	0
Mix30	60	30	10	0
Mix40	60	40	0	0

† WTR, Water treatment residuals

Table 2. Target contaminant dosing.

Analyte	Target concentration (µg/L)
Total Copper	20.0
Dissolved Copper	7.0
Total Zinc	150.0
Dissolved Zinc	50

Table 3. Concentration, load, and reduction of copper and zinc.

Mix	Storm A						Storm B					
	C_{in} (µg/L)	C_{eff} (µg/L)	ΔC (%)	L_{in} (mg)	L_{eff} (mg)	ΔL (%)	C_{in} (µg/L)	C_{eff} (µg/L)	ΔC (%)	L_{in} (mg)	L_{eff} (mg)	ΔL (%)
Total Copper												
Mix15	19	5.5 ±0.3	70.8 ±1.6	7.8	2.2 ±0.1	71.5 ±1.5	17.6	6.2 ±0.4	64.6 ±2.0	15.1	5.3 ±0.2	64.7 ±1.4
Mix20		4.6 ±0.8	75.7 ±4.3		1.7 ±0.2	77.7 ±2.9		6.0 ±0.4	65.8 ±2.5		5.0 ±0.3	67.0 ±2.0
Mix30		5.6 ±0.8	70.7 ±4.4		2.3 ±0.3	70.6 ±3.4		6.2 ±0.9	64.6 ±5.0		5.4 ±0.6	64.4 ±4.0
Mix40		4.2 ±0.5	77.8 ±2.5		1.8 ±0.1	77.4 ±0.9		6.0 ±0.8	65.6 ±4.6		5.2 ±0.3	65.5 ±1.7
Dissolved Copper												
Mix15	11.7	3.8 ±0.3	67.3 ±2.8	4.8	1.5 ±0.1	68.1 ±2.4	7.4	4.8 ±0.2	34.8 ±3.2	6.3	4.1 ±0.0	34.9 ±0.6
Mix20		3.2 ±0.4	72.4 ±3.4		1.2 ±0.0	74.7 ±0.9		4.5 ±0.4	39.9 ±4.7		3.7 ±0.2	42.0 ±3.0
Mix30		4.0 ±0.6	65.6 ±5.3		1.7 ±0.2	65.7 ±3.7		5.0 ±0.5	31.8 ±7.4		4.4 ±0.4	31.3 ±5.7
Mix40		3.5 ±0.6	69.7 ±5.2		1.5 ±0.2	69.2 ±3.3		5.0 ±0.5	32.8 ±7.0		4.3 ±0.1	32.4 ±1.7
Total Zinc												
Mix15	180	4.2 ±0.5	97.6 ±0.3	74.1	1.7 ±0.2	97.7 ±0.3	158	4.5 ±0.6	97.2 ±0.4	135.4	3.9 ±0.6	97.1 ±0.5
Mix20		4.2 ±0.5	97.6 ±0.3		1.6 ±0.2	97.8 ±0.3		4.8 ±1.0	97.0 ±0.6		4.0 ±0.9	97.1 ±0.7
Mix30		4.0 ±0.0	97.8 ±0.0		1.7 ±0.1	97.8 ±0.1		4.8 ±0.5	97.0 ±0.3		4.1 ±0.3	97.0 ±0.2
Mix40		4.0 ±0.0	97.8 ±0.0		1.7 ±0.1	97.7 ±0.2		4.5 ±0.6	97.2 ±0.4		3.9 ±0.6	97.1 ±0.4
Dissolved Zinc												
Mix15	154	4.0 ±0.0	97.4 ±0.0	63.4	1.6 ±0.1	97.5 ±0.2	98	4.0 ±0.0	95.9 ±0.0	84	3.4 ±0.2	95.9 ±0.2
Mix20		4.0 ±0.0	97.4 ±0.0		1.5 ±0.2	97.6 ±0.2		4.0 ±0.0	95.9 ±0.0		3.3 ±0.3	96.0 ±0.4
Mix30		4.0 ±0.0	97.4 ±0.0		1.7 ±0.1	97.4 ±0.1		4.0 ±0.0	95.9 ±0.0		3.5 ±0.1	95.9 ±0.2
Mix40		6.0 ±4.0	96.1 ±2.6		2.5 ±1.7	96.0 ±2.8		4.0 ±0.0	95.9 ±0.0		3.5 ±0.3	95.9 ±0.3

C_{in}: inflow concentration; C_{eff}: effluent concentration; ΔC: concentration reduction; L_{in}: input load; L_{eff}: effluent load; ΔL: load reduction.

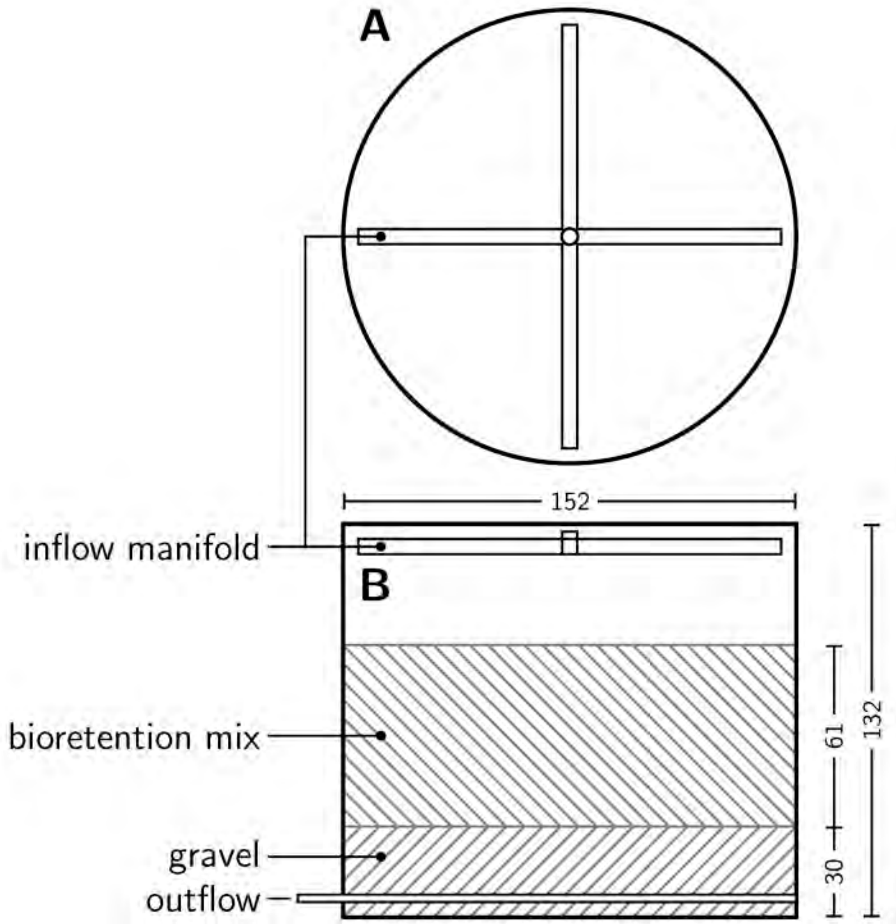

Figure 1. Top (A) and side view (B) schematic of the bioretention cell. All units are in centimeters.

Community Wastewater Infiltration at 69o Northern Latitude – 25 Years of Experience.

Petter Jenssen, Norwegian University of Life Sciences

ABSTRACT

When Bardu municipality located at 690 northern latitude in Norway were to renew their wastewater treatment facility in the early 80«s they chose to pump the sewage effluent from the 5000 inhabitants of Setermoen into a nearby glaciofluvial sand and gravel deposit. Initially the system consisted of two open sedimentation basins succeeded by three 2m deep open v-shaped alternating infiltration basins. The deep basins were chosen so that the surface could freeze while the water would still infiltrate below the ice. In year 2000 the municipality decided to install garbage grinders in all homes. This increased the organic load to the system and a new sedimentation basin succeeded by a simple surface trickling system was constructed up front of the existing system. The unsaturated zone below the basins is 7m. Since the startup in 1987 groundwater has been pumped regularly from a well between the infiltration basins. A large groundwater survey (1995 -1998) showed that this well gave representative values of the treated water. The overall treatment performance has been 85-90% for COD, 60-70% for total nitrogen (N) and 99% for phosphorus (P). The water meets European standards for swimming water with respect to indicator bacteria. Despite an average annual temperature of +0.7oC nitrification with subsequent denitrification can explain the high N-removal. Under each basin the capacity for P-removal is estimated to last 14 years. The system has saved the municipality an estimated 45 million NOK over 25 years compared to investment and operation of a conventional mechanical/chemical treatment system.

An Environmental Impact Study on the Manufacture, Production, and Transport of Septic Systems.

Jessica Barringer, Infiltrator Systems Inc.

ABSTRACT

Global concern is growing over natural resource consumption and climate change. Many governments, companies, and industries are taking action to reduce the environmental footprint associated with material and product manufacture and processing. Both natural resource consumption and greenhouse gas emissions are being monitored closely as resource shortages and emissions continue to rise globally.

Onsite wastewater treatment systems have historically been composed of concrete septic tanks and stone/pipe drainfields. However, the processes and materials used to manufacture conventional systems use a large amount of resources (aggregate, water, fuel, electricity) and emit a large amount of CO_2. Alternatively, other materials have been increasingly substituted for conventional materials, including recycled thermoplastic septic tanks and chambers. These materials have qualitatively been considered more environmentally friendly, but no quantitative comparison has been evaluated in regards to resource consumption and carbon emissions.

Therefore, the environmental impacts of both conventional septic systems and systems using recycled thermoplastics were evaluated. A conventional septic system was defined as a precast septic tank and gravel/pipe drainfield. Infiltrator Systems Inc. (Infiltrator) products, the IM-1060 and Quick4 Standard chambers, were used to represent recycled thermoplastic systems. Water consumption, electricity consumption, fuel consumption, and carbon emissions were evaluated through raw material production, product manufacturing and transportation for both systems.

It was determined that even when transporting the recycled thermoplastic systems 1000 miles and conventional systems only 30 miles, the recycled systems reduced electricity consumption by 85% (5296 kWh saved), fuel consumption by 56% (16 gal saved), water consumption by 92% (393 gal saved), and carbon emissions by 81% (1190 kg C saved). When compared to the total number of septic systems installed each year (26.1 million in 2007, USEPA), this could amount to a total savings of 138 billion kWh of electricity, 417.6 million gallons of fuel, 10.3 billion gallons of water, and 31 million tons of carbon if every septic system was composed of recycled thermoplastics rather than conventional materials.

Fate and Transport of Phosphorus Beneath Mounded Septic Drainfields.

Gurpal Toor, University of Florida

ABSTRACT

In most aquifers and watersheds in the world, the contribution of phosphorus (P) from septic systems to groundwater is largely not quantified. In areas with sandy soils and shallow water table such as in Florida, the drainfield of septic systems is constructed in raised beds (commonly known as mounds).

Treatment of Trace Organic Compounds in Common Onsite Wastewater Systems

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ABSTRACT

Onsite wastewater systems have historically been relied on to treat conventional pollutants and pathogens in a fashion similar to that expected from centralized wastewater systems. However, based on the potential occurrence of, and effects from, contaminants of emerging concern in wastewaters, onsite systems as well as centralized systems need to account for this in system design and use. One group of contaminants involves organic compounds such as those associated with consumer product chemicals and pharmaceuticals, which are collectively referred to as trace organic compounds due to their very low levels relative to other pollutants. The question being confronted today is how best to account for trace organics in onsite system design and use while also achieving other goals such as system simplicity, limited operation and maintenance requirements, low cost, and sustainability. As highlighted in this paper, there are a large number of trace organic compounds that can be present in onsite wastewaters and they have different properties, can be present at different frequencies of occurrence and concentrations, and have different susceptibilities to treatment in onsite wastewater systems. In general, trace organic compounds normally should not require additional considerations beyond those for conventional pollutants and pathogens (e.g., nitrogen or bacteria and virus) during design and use of onsite wastewater systems. That said, there are situations where trace organics could be a serious concern warranting special consideration in system design and use. In this paper, the frequency of occurrence of trace organic compounds and the range of concentrations encountered are highlighted. An evolving approach is outlined that could help assess the likelihood of occurrence and levels of trace organics along with the treatment anticipated in different onsite wastewater systems and assimilation conditions.

INTRODUCTION

Onsite wastewater systems have been used for more than a century throughout the United States to handle small wastewater flows from homes, businesses, and small towns. Systems were initially selected to achieve simple goals such as wastewater disposal with little to no operation and maintenance at low cost. Over time, the inherent capabilities and benefits of onsite systems were increasingly recognized and the goals evolved to include effective treatment of wastewater while also enabling recharge of local water resources, recovery and reuse of water, organic matter, and nutrients, lower consumption of energy and chemicals, and providing infrastructure that might be more robust and resilient to natural disasters and climate change. As the goals have evolved, so have the challenges being confronted. Onsite wastewater systems historically had to treat conventional pollutants and pathogens. During the past decade however, concerns have arisen and grown over contaminants of emerging concern such as those associated with consumer product chemicals (e.g., Triclosan, bisphenol-A), pharmaceuticals (e.g., ibuprofen, sulfamethoxazole), and flame retardants (e.g., perfluorooctane sulfonate) (Conn et al. 2006). Based on the potential occurrence of and effects from these organic compounds in wastewater, there has been a growing need for increased understanding concerning their occurrence and fate in common onsite wastewater treatment systems (e.g., Fig. 1).

Research and educational efforts during the past decade or more have helped advance the science and engineering of onsite system selection, design, and use. Among the efforts initiated in the United States during the 1990s, the Small Flows Program was established at the Colorado School of Mines (CSM) in Golden, Colorado. At CSM, research has been carried out to: 1) determine the flow and composition of modern onsite wastewater streams, 2) evaluate the performance dynamics of bioreactors and biofilters, including their integration with soil-based unit operations, 3) evaluate the performance of decentralized systems utilizing membrane bioreactors, and 4) develop mathematical models and decision support tools (Siegrist et al. 2013). One area of emphasis within these elements has been concerned with the occurrence of contaminants of emerging concern in wastewaters being handled by onsite systems and the treatment efficiency achieved by common unit operations and systems. This paper highlights some of the research carried out concerning trace organic compounds and presents ideas about how to account for their occurrence and fate during onsite system selection, design and use.

OCCURRENCE AND FATE OF TRACE ORGANIC COMPOUNDS

<u>Occurrence of Trace Organics in Onsite Wastewaters.</u> Quantitative understanding of water use and wastewater characteristics is important for proper system selection and design. Until recently much of the available characterization data were for conventional pollutants and pathogens and based on studies done decades ago. Recent and ongoing CSM research projects have been focused on advanced characterization of modern waste streams.

In an early study, characterization data for trace organic compounds were obtained through field monitoring at 30 sites in Colorado (Conn et al. 2006). In a subsequent study, field monitoring was completed at 17 domestic sites in three regions of the United States (Lowe et al. 2009, Conn et al. 2010a). A specialized apparatus was fabricated to collect 24-hr composite samples of raw wastewater and septic tank effluent (STE) during each season of the year. Analyses were made for conventional wastewater parameters (e.g., flow, pH, $cBOD_5$, nutrients, microorganisms) and a suite of trace organic compounds. Example results for the frequency of occurrence and concentrations measured for consumer product chemicals, pharmaceuticals, and flame retardants are presented in Tables 1 to 3.

This CSM research, along with work done by others (e.g., Hinkle et al. 2005, Swartz et al. 2006, Teerlink et al. 2012), has revealed that a wide range of trace organic compounds including consumer product chemicals, pharmaceuticals, and other compounds can occur in wastewaters that are often treated in onsite and decentralized systems (Tables 1 to 3). These compounds include caffeine, methylphenol, and the antimicrobial Triclosan, endocrine-disrupting detergent metabolites such as 4-nonylphenol and 4-nonylphenolethoxylates, natural and synthetic hormones such as 17-β-estradiol and estrone, and non-prescription pharmaceuticals such as ibuprofen and acetaminophen. Results from the limited characterization data for non-residential sources such as food establishments, convenience stores, schools, and clinics have revealed that they can have unique and potentially higher-strength wastewater composition regarding trace organic compounds (Fig. 2). The reported concentrations of trace organic compounds in onsite wastewaters range over more than three orders of magnitude, from less than 1 μg/L to over 1000 μg/L. Concentrations can vary widely between sources and with time at the same source. This variability is due to the fact that with a single source or small number of sources, the influent wastewater composition is highly dependent on the water using activities and chemical usage within a source and at a certain time (Conn et al. 2006).

<u>Treatment and Fate of Trace Organics in Common Onsite Systems.</u> Onsite and decentralized systems involve unit operations that can be combined to achieve up to tertiary treatment levels with disinfection. Different types of systems can enable different discharge and reuse options. Research carried out at CSM has investigated the performance of contrasting systems through

laboratory experiments, field-testing at the Mines Park Test Site located on the CSM campus, and full-scale systems monitoring. This work has focused in part on the treatment of trace organic compounds in confined unit operations (e.g., a textile biofilter) and natural system operations (e.g., a network of soil infiltration trenches).

In the CSM project where trace organic compounds were characterized in wastewaters at 30 sites (see Table 1) additional monitoring was completed to assess their removal in septic tanks, biofilters, and constructed wetlands (Conn et al. 2006). Removal efficiencies were observed to range from <1% to >99%, with the efficiency dependent on the properties of the trace organic compounds and the removal processes operative in the treatment unit (Fig. 3). For example, compared to anaerobic treatment in a septic tank, additional aerobic treatment in a textile biofilter enhanced the removal of trace organic compounds that were susceptible to aerobic biodegradation. In a companion project completed at the Mines Park Test Site, this relationship was further revealed, as the removal efficiency in a textile biofilter was generally greater than in a septic tank alone (Table 4) (Conn *et al.* 2010b).

Onsite and decentralized systems often involve use of soil as a final unit operation for treatment and assimilation of the effluent-derived water into the local hydrologic regime. CSM research in this area has focused on two onsite system approaches: 1) effluent dispersal into a soil profile using shallow trenches outfitted with infiltration chambers (e.g., Lowe and Siegrist 2008) and 2) effluent dispersal into the plant rhizosphere using drip tubing with pressure-compensating emitters (e.g., Siegrist et al. 2014). In addition to revealing the time-dependent and dynamic interaction of unit hydraulics and purification processes for conventional pollutants and pathogens, these studies also provided new insights into the fate of trace organic compounds. For example, to understand their fate in a soil treatment unit (Fig. 1), a controlled field experiment was completed at the Mines Park Test Site at CSM (Conn et al. 2010b). The effluents from a septic tank or a textile biofilter (Table 4) were applied to an Ascalon sandy loam soil and the soil pore water was periodically sampled at 60, 120, and 240 cm below the soil infiltrative surface. The subsurface depth profiles for several constituents are shown in Fig. 4. Purification of trace organic compounds (e.g., caffeine, nonylphenols, Triclosan) in a soil treatment unit principally occurs by sorption and biodegradation processes. Achieving high removal efficiency for a particular organic compound thus depends on the properties of the compound as well as the process conditions present in the soil treatment unit (Conn et al. 2010b). Infiltration of wastewater effluents like STE enhance the treatment ability of native soil by generating a biozone at the infiltrative surface and stimulating unsaturated flow in an underlying aerobic soil profile. Biozone genesis has been characterized to include three processes: a) biofilm formation, b) biomat development, and c) humic substance-like material development (Siegrist 2007, McKinley and Siegrist 2010). As a result, a soil treatment unit characterized by unsaturated flow under aerobic conditions through some depth to groundwater (e.g., 90 cm) can achieve very high removal efficiencies for many compounds. For example, during the research at the Mines Park Test Site, caffeine and Triclosan in septic tank or textile filter unit effluents were completely removed by 60-cm depth presumably through aerobic biotransformation (Fig. 4) (Conn et al. 2010b).

ACCOUNTING FOR TRACE ORGANICS IN SYSTEM DESIGN AND USE

Onsite and decentralized systems have historically been relied on to treat conventional pollutants and pathogens and thereby protect public health and environmental quality at the local to watershed scale. There is now a growing interest in understanding how system design and use can and should account for trace organic compounds that could be viewed as micropollutants. This is by no means a simple task, as there are a large number of trace organic compounds that can be present in wastewaters from different sources, with different properties and potential health and environmental effects, with different frequencies of occurrence and concentrations (often extremely low levels), with different removal efficiencies in unit operations and systems and different fates during assimilation under the local environmental conditions.

A first question for a particular project where an onsite wastewater system may be implemented involves gaining some sense of the likelihood of occurrence of different trace organic compounds and at what concentrations. Knowledge of the water using activities within a source can provide initial information regarding the likely types and levels of trace organic compounds that might occur and need to be handled by an onsite or decentralized wastewater system. Figure 5 has been developed to portray the factors that need to be considered in this regard. For example, sources whose wastewater primarily originates from restroom use (e.g., roadside convenience stores, campgrounds, parks) will likely have elevated levels of compounds such as fecal sterols and excreted pharmaceuticals in their wastewaters. Conn et al. (2006) found that gas station convenience store wastewaters had the highest concentrations of 14 pharmaceuticals as compared to other sources. Sources with intense and frequent cleaning practices (e.g., human or animal institutions, food establishments, vacation residences) will likely have elevated levels of consumer product chemicals in their wastewaters. For example, veterinary hospital wastewater, which mainly originates from washing and disinfecting practices, had high levels of surfactant metabolites at concentrations up to 20 times greater than other sources (Conn et al. 2006). Actual wastewater compositions will depend on the specific products in use within the source and the recent chemical- and water-using activities, all of which can differ between sources and at the same source over time.

Achieving treatment of trace organic compounds (e.g., caffeine, nonylphenols, Triclosan) in an onsite wastewater system is highly dependent on the properties of the compound (e.g., hydrophobicity, volatility, biodegradability) and the processes occurring in the unit operations that comprise the system (e.g., septic tank, textile biofilter, constructed wetland, soil treatment system). Figure 6 has been prepared to help capture the relevant processes and parameters affecting treatment efficiency for different types of compounds under different conditions. The key processes included in Figure 1 and important to treatment for most trace organic compounds include: biodegradation, sorption, and volatilization. In plant-based systems (e.g., constructed wetlands or landscape drip dispersal) other processes may also play a role (e.g., photolysis or plant uptake). Achieving high removal efficiency for a particular trace organic compound thus depends on the properties of the compound as well as the process conditions present in a treatment unit operation (Fig. 6) (Conn et al. 2010b). For example, within a common onsite wastewater system high removal is expected for a compound such as dichlorobenzene due to its sorption affinity to organic matter and its volatility. In contrast, negligible removal is expected for a compound such as carbamazepine, which is not volatile or biodegradable, and does not sorb to organic matter (Fig. 6).

Beyond treatment in a confined unit operation and unconfined soil treatment unit, the treated effluent-derived water is eventually assimilated into the subsurface hydrologic regime (both deeper unsaturated zone and groundwater zone) (Fig. 1). During this assimilation, further treatment of any remaining trace organic compounds can occur during transport away from a site and prior to reaching a water supply well or nearby surface water. This treatment occurs by dilution and attenuation processes (e.g., sorption, biodegradation), which can be generically represented by a dilution attenuation factor (DAF) (USEPA 1996). The DAF is defined as the ratio of the concentration of a trace organic compound in soil pore water to its concentration at a specified location (e.g., at a nearby drinking water supply well). Values of DAF depend on circumstances including subsurface conditions and transport distances away from the onsite treatment system to a location of interest (e.g., drinking water supply well). DAF values that have been developed for assessment of contaminated soils have varied from 1 (with no dilution or attenuation) to values of 100. Conn et al. (2010b) applied this DAF approach to assessment of 4-nonylphenol fate below an onsite wastewater system including subsurface infiltration of STE into infiltration trenches installed in Ascalon sandy loam soil with an unsaturated, aerobic soil profile to more than 2.4 m depth. Conn et al. (2010b) reported that the concentration of 4-nonylphenol in the subsurface would only exceed a USEPA toxicity-based water quality criteria if the location of interest were soil pore water at 60-cm depth with a DAF of 1.

The degree to which a common onsite system might need to account for trace organics depends in large part on the likelihood of occurrence of a trace organic compound that could exert adverse effects under the site conditions and land use surrounding the location of the onsite wastewater treatment system. In general, compared to conventional pollutants and pathogens (e.g., nitrogen or bacteria and virus) trace organic compounds are probably not a dominant concern for the vast majority of common onsite wastewater systems. This is due to a number of reasons including the relative sporadic occurrence and/or extremely low concentrations present, the treatment processes occurring in onsite systems with an inherent ability to remove trace organic compounds, and the attenuation that occurs prior to treated effluent-derived water reaching a potential receptor. However, there are situations where trace organics will be a serious concern warranting special consideration in system selection, design and use. For example, an onsite wastewater treatment system serving a convenience store or medical clinic where high concentrations of trace organics might be anticipated would require special consideration if it were located in a suburban development with shallow private wells used for drinking water supply. In another example, a coastal retirement community consisting of aging onsite wastewater systems may require special consideration, especially if the groundwater recharges adjacent surface waters used for recreation or shellfishing. Swartz et al. (2006) reported the presence of natural and synthetic hormones in shallow groundwater in Cape Cod, MA, at concentrations similar to those measured in the septic tank wastewater of the overlying onsite wastewater system. For these situations the onsite wastewater system might necessarily have to include more advanced treatment than that provided by a septic tank; for example using an aerobic treatment unit, recirculating biofilter, or constructed wetland before soil-based treatment and subsurface assimilation.

The ultimate disposition of the effluent from an onsite wastewater system and the context for its use are of major importance with respect to the potential level of concern for system selection, design, and use. Most onsite wastewater systems accomplish partial treatment in a confined unit operation before discharging the effluent into the subsurface for final treatment and assimilation into the hydrologic regime. However, there is growing interest in onsite water reclamation and reuse including use of treated effluents for flushing toilets and irrigating landscapes. To enable safe employment of these reuse schemes, source separation approaches are being advocated (e.g., graywater separation for treatment and reuse) and advanced treatment unit operations can be used if needed (e.g., membrane bioreactors). There is also growing interest in direct urine diversion and recovery to enable capture of nutrients for use in agronomic fertilization.

Accounting for the occurrence and the treatment of trace organics in these and other approaches to onsite water reclamation and reuse are beyond the scope of this paper.

Natural and engineered treatment technologies, such as those mentioned in this paper as well as technologies employed in municipal wastewater treatment plants, can effectively remove many trace organic compounds from wastewater. Removal from wastewater, however, often does not equate to removal from the environment. For example, removal may be through sorption to particulates, such as settling solids in septic tanks. This can result in accumulation of hydrophobic compounds in these solids, which are periodically removed, dewatered, and often land-applied for "final" disposal. In biosolid-amended agricultural soil, trace organic compounds including nonylphenol and Triclosan that were present in the soil also bioaccumulated in terrestrial organisms (earthworms) (Kinney et al. 2008).

Many of the trace organic compounds of potential concern enter wastewater via biogenic sources or from chemicals used in a home, business or institution. Qualitatively, compounds such as hormones, sterols, and pharmaceuticals primarily originate from biogenic sources (i.e. human urine and feces). Other compounds such as surfactant metabolites and fragrances primarily originate from washing and cleaning activities. For certain trace organics, which can be present at low but meaningful concentrations, it can be challenging to achieve high removal efficiencies in unit operations within common onsite systems. And as just noted, even when systems effectively remove trace organics from wastewater effluents they can accumulate in residuals and be re-introduced into environmental systems where they might cause adverse effects. This exemplifies the importance of care and caution in using consumer product chemicals, pharmaceuticals, and other trace organic compounds and highlights the value in avoiding, as far as possible, the use of and release of chemicals during water using activities that generate wastewater for treatment. The use of chemical-free products may help reduce the occurrence and levels of these types of compounds in onsite wastewater effluents.

CONCLUSIONS

As highlighted in this paper, there are a large number of trace organic compounds that can be present in onsite wastewaters. In general, compared to conventional pollutants and pathogens (e.g., nitrogen or bacteria and virus) trace organic compounds are probably not a dominant concern for the vast majority of onsite wastewater systems. In this paper, an approach is proposed to help assess the likelihood of occurrence and level present for trace organics in wastewaters from different sources along with the treatment anticipated in different treatment systems and varied assimilation conditions. Beyond the scope of this paper are source separation approaches and advanced treatment operations designed to enable onsite water reuse and resource recovery. Further research is needed to broaden and advance the level of understanding and fully develop and test the approach and concepts outlined here.

ACKNOWLEDGEMENTS

Numerous individuals and organizations have contributed to the research highlighted in this paper. The efforts of past and current students and staff at CSM are gratefully acknowledged including: Chris Bellona, Christiane Hoppe, Kathryn Lowe, Jim McKinley, Nate Rothe, Jill Tomaras, and Mia Tucholke. Contributing faculty have included Dr. Jörg Drewes and John McCray. Collaborators at the U.S. Geological Survey have included Dr. Larry Barber and others in the Toxic Substances Hydrology Program. Program funding has been acquired through CSM and its Research Foundation, the USGS National Institutes of Water Research and Water Environment Research Foundation along with contracts and philanthropic grants from private industry.

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Figure 1. Illustration of an onsite wastewater treatment system and the relevant processes affecting the source concentrations and removal of trace organic compounds.

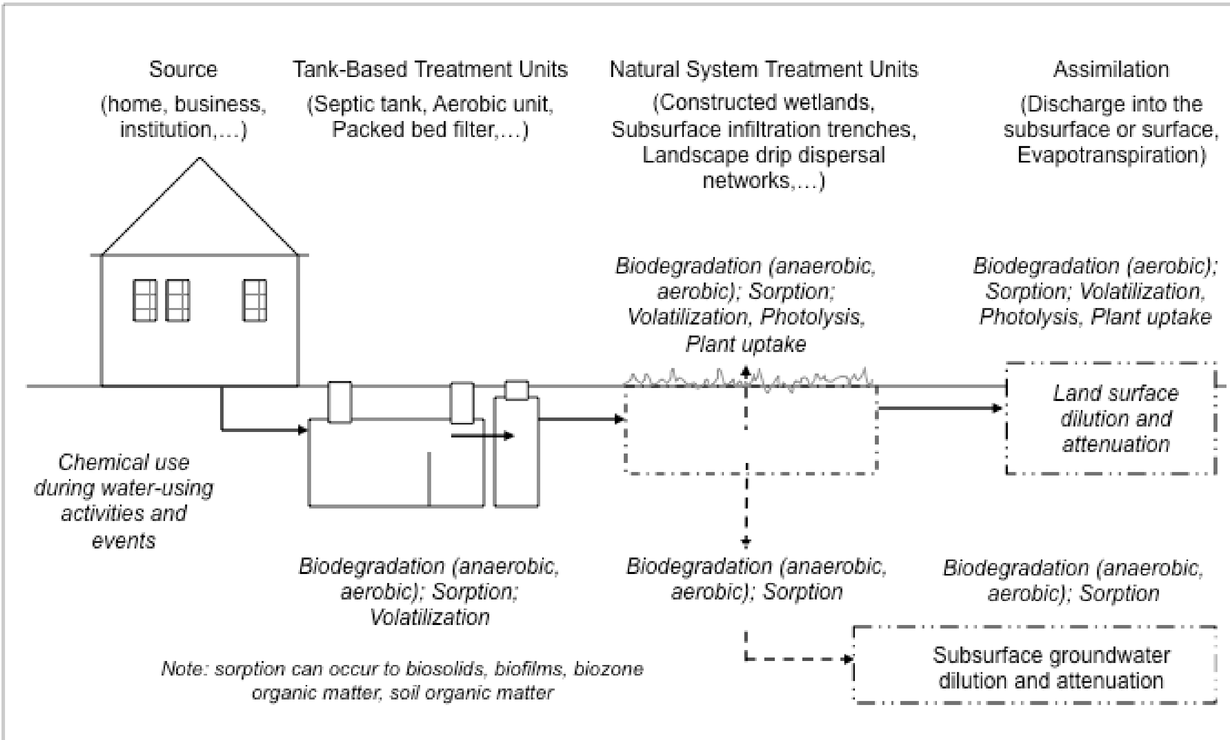

Figure 2. Illustration of the variation in the concentrations of trace organic compounds found in the total wastewater generated from residential, commercial or institutional sources (Conn et al. 2006).

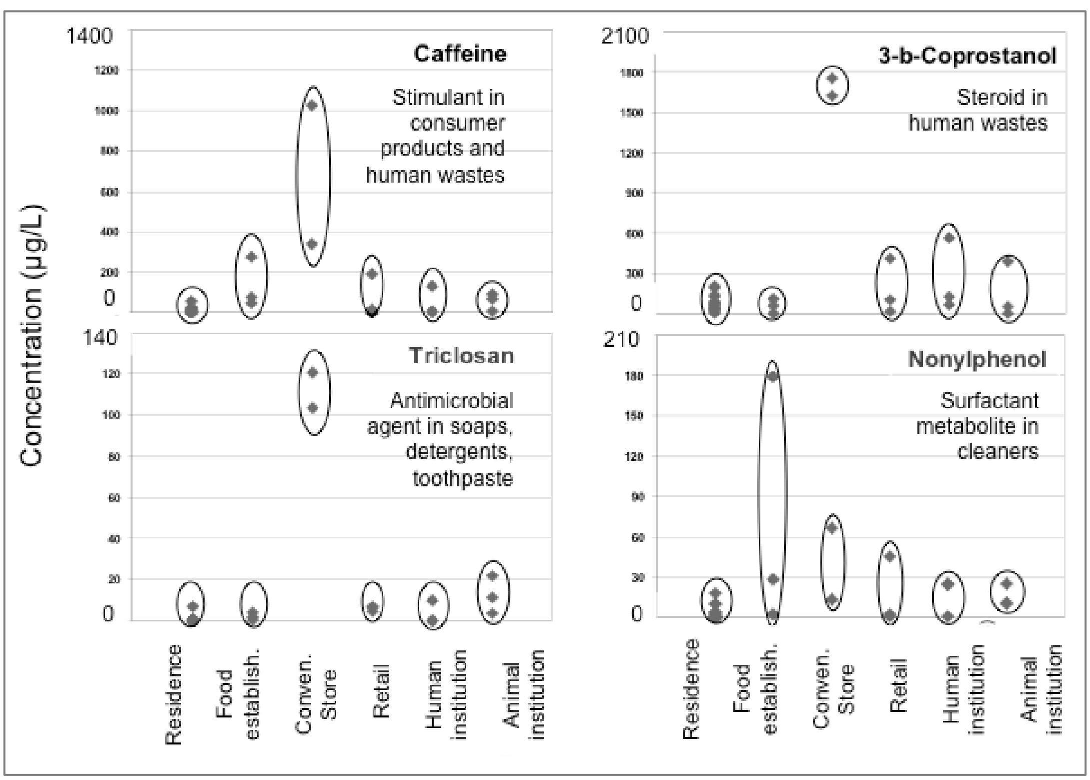

Figure 3. Illustration of the removal efficiency for caffeine in wastewaters treated by different types of onsite treatment operations that have different transformation and removal processes (Conn et al. 2006).

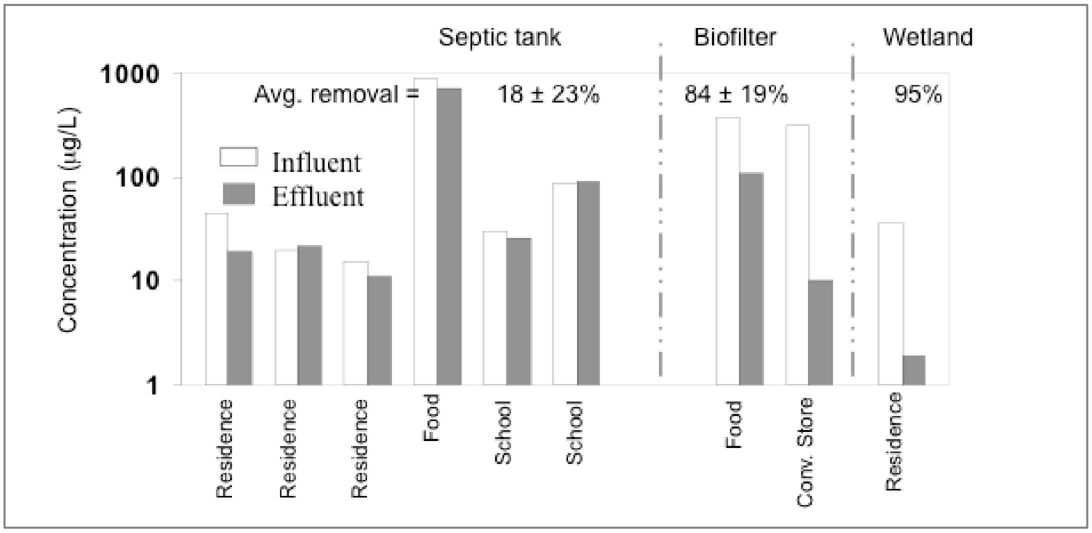

Figure 4. Concentrations of selected constituents in vadose zone pore water at three different depths below the soil infiltrative surface at the Mines Park Test Site (Conn et al. 2010b). (Note: STE = septic tank effluent; TBE = textile biofilter effluent; Water = City of Golden tap water; Shown at 0 cm is the average effluent concentration; for visualization purposes, values less than the reporting level (RL) are shown as ½ of the RL and lines connect the data points with depth; the 2 cm/d data points represent the average from 3 test cells each sampled 3 times; the 8 cm/d data points represent the average from 1 test cell sampled 3 times.)

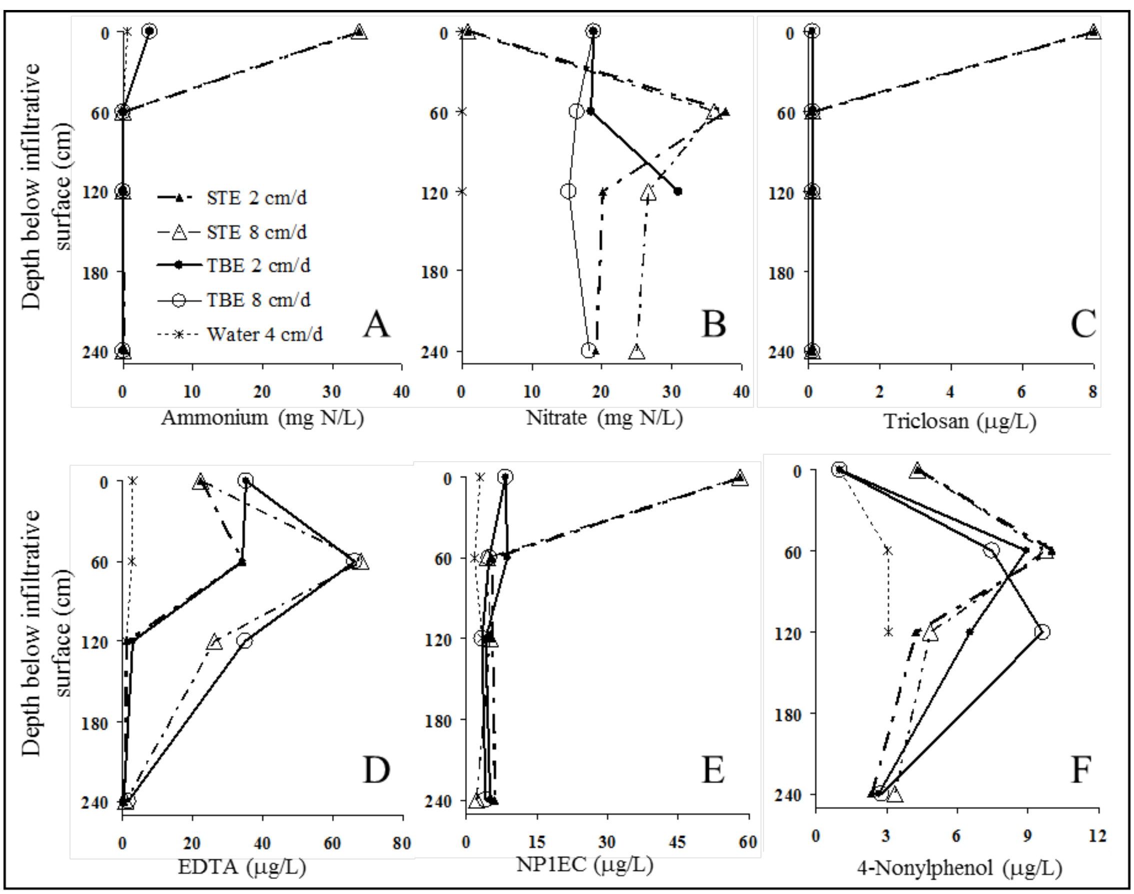

Figure 5. Decision diagram for assessing the likelihood of occurrence and anticipated concentrations of trace organic compounds in total wastewaters often treated by common onsite wastewater systems. (Note: This diagram is preliminary and provided for illustrative purposes only. It is intended for total wastewater streams from common sources and does not address source modification schemes such as graywater separation or urine diversion.)

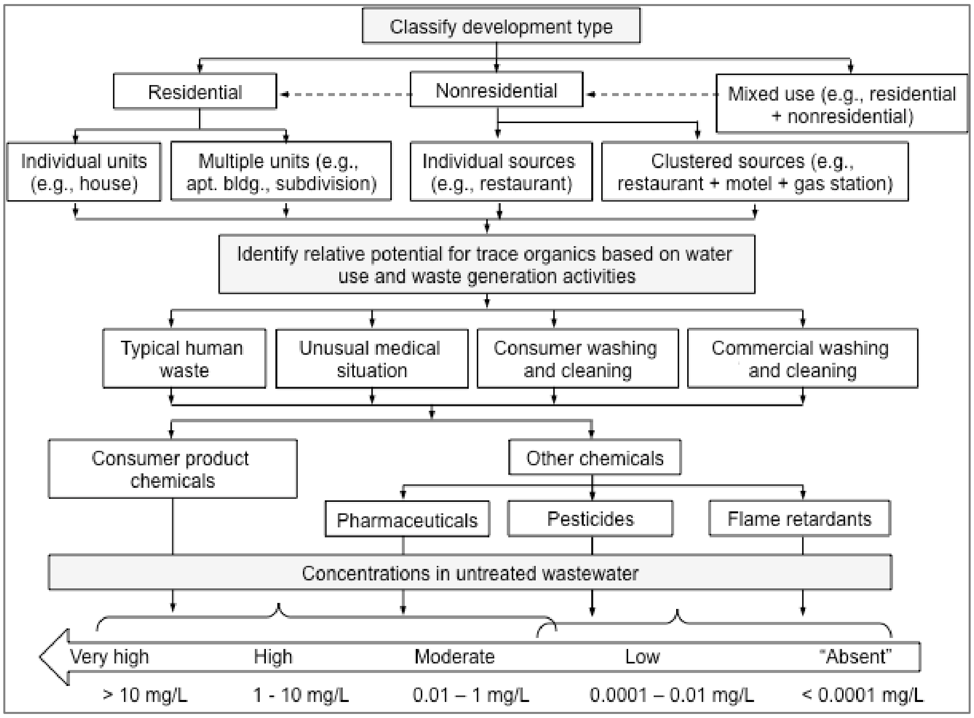

Figure 6. Decision diagram for assessing the anticipated treatment efficiency and attenuation of trace organic compounds in common onsite wastewater systems. (Note: This diagram encompasses common onsite wastewater systems (e.g., septic tank, aerobic treatment unit, intermittent filters, soil infiltration units) and does not address advanced treatment operations (e.g., nanofiltration, membrane systems). Half-life is for aerobic biodegradability, K_{OW} is the octanol-water partition coefficient (-) and K_H = Henry's constant (atm•L/mol). DAF = dilution-attenuation factor.)

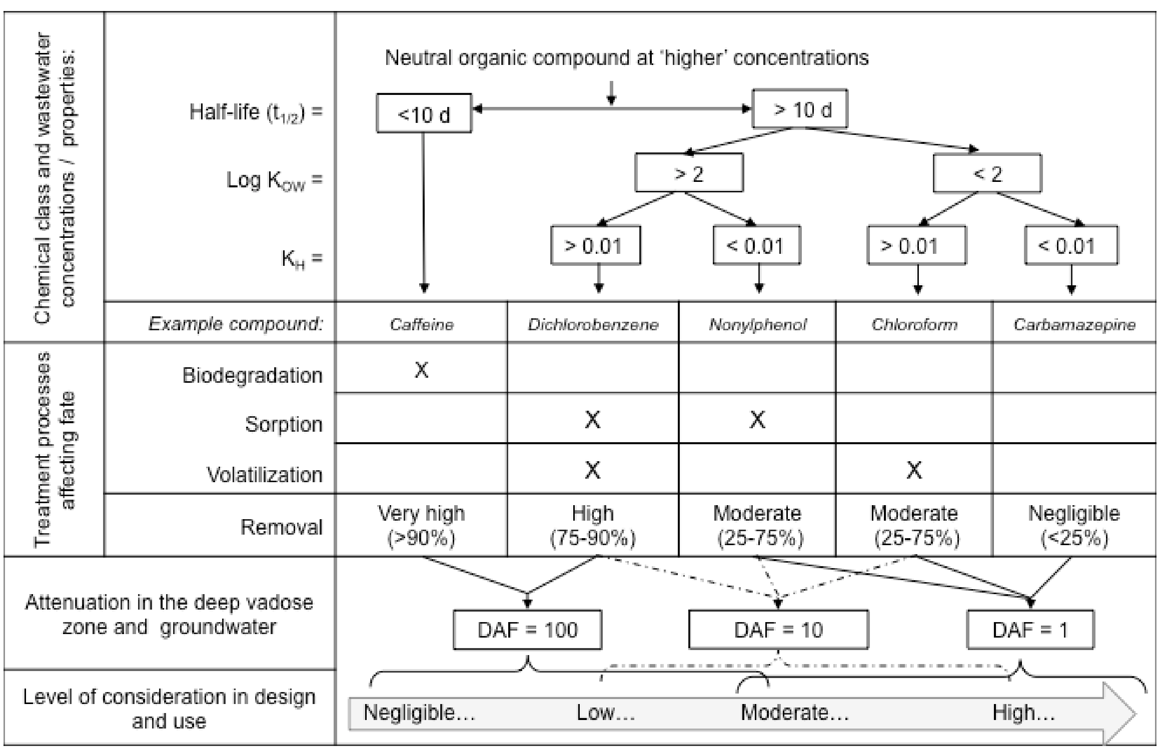

Table 1. Frequency of occurrence and concentration of trace organic compounds associated with consumer product chemicals in wastewaters from 30 small residential, commercial, and institutional sources in Colorado (Conn et al. 2006).

Organic compound	Use	Detection frequency	Concentration range (µg/L)
Caffeine	Stimulant	100%	0.5 – E 9,300 [1]
Coprostanol	Animal sterol	100%	0.5 – E 7,100
Cholesterol	Animal sterol	100%	0.5 – E 2,200
Ethylenediaminetetraacetic acid (EDTA)	Metal chelation	100%	0.5 – 1,700
4-Methylphenol	Disinfectant	98%	0.5 – E 4,500
4-Nonylphenolethoxycarboxylates (NPEC)	Surf. metabolite	95%	2 - 320
Nitrilotriacetic acid (NTA)	Metal chelation	82%	0.5 - 130
4-Nonylphenol	Surf. metabolite	77%	2 – 340
4-Nonylphenolethoxylates (NPEO)	Surf. metabolite	75%	2 – 170
5-chloro-2-(2,4-dichlorophenoxy)phenol (Triclosan)	Antimicrobial agent	68%	0.5 – 82

[1] E = estimated value (concentration exceeded maximum value on standard curve).

Table 2. Frequency of occurrence and concentration of trace organic compounds in wastewaters from 17 residential sources in three regions of the United States (Lowe et al. 2009).

Compound	Use	RL[1] (ug/L)	Detection frequency (%)	Concentration (µg/L)	
				Median	Max.
Bisphenol A	Plasticizer	0.2	1/12 (8)	18	18
Caffeine	Stimulant	0.2	13/13 (100)	93	E 1800 [2]
EDTA	Metal chelating agent	0.1	4/4 (100)	33	E 720
NTA		0.02	1/4 (25)	4.5	4.5
4-Nonylphenol	Surfactant metabolite	2	9/13 (69)	6.8	66
NP1EO		1	13/13 (100)	7.5	23
Triclosan	Antimicrobial	0.2	13/13 (100)	19	230

[1] RL = reporting limit. [2]E = estimated value (concentration exceeded maximum value on standard curve).

Table 3. Frequency of occurrence and concentration of pharmaceuticals, pesticides, and flame retardants in residential wastewaters (Lowe et al. 2009).

Compound	Use	RL (µg/L)	Detection frequency (%)	Concentration (µg/L)	
				Median	Max.
Clofibric acid	Lipid regulating	0.1	0/15 (0)	nd [1]	nd
Dchlorprop	Pesticide	0.1	0/15 (0)	nd	nd
Diclofenac	Anti-inflammatory	0.1	0/15 (0)	nd	nd
Fenfibrate	Lipid regulating	0.2	0/15 (0)	nd	nd
Gemfrizol	Lipid regulating	0.1	0/15 (0)	nd	nd
Ibuprofen	Analgesic	0.1	5/15 (33)	22.1	E 146
Ketoprofen	Analgesic	0.1	0/15 (0)	nd	nd
Mecoprop	pesticide	0.1	0/15 (0)	nd	nd
Naproxen	Analgesic	0.1	2/15 (13)	E 178	E 178
Phenacetine	Analgesic	0.2	0/15 (0)	nd	nd
Salicylic acid	Anti-inflammatory	0.1	13/15 (87)	E 47.5	E 208
Tris(2-chloroethyl)phosphate	Flame retardant	0.2	0/15 (0)	nd	nd
Tris(2-chloroisopropyl)phosphate	Flame retardant	0.2	0/15 (0)	nd	nd
1,3-dichloro-2-propanol phosphate	Flame retardant	0.2	0/15 (0)	nd	nd

[1] nd = Not detected.

Table 4. Concentrations of trace organic compounds in the effluents from a septic tank versus a textile biofilter used to treat wastewater from an 8-unit apartment building (Conn et al. 2010b).

Compound	Units	Septic tank effluent	Textile biofilter effluent
Dissolved organic carbon	mg/L	30 (8.4)[1]	16 (4.2)
Ammonium	mg-N/L	34 (7.5)	3.8 (1.1)
Nitrate	mg-N/L	0.85 (0.48)	19 (3.8)
Caffeine	µg/L	34 (8.7)	0.87 (0.49)
EDTA	µg/L	24 (1.0)	33 (13)
NTA	µg/L	3.7 (2.3)	4.0 (1.9)
4-Nonylphenol	µg/L	3.3 (1.4)	<RL[2] of 2
NPEC	µg/L	63 (23)	7.3 (3.6)
NPEO	µg/L	1.6 (0.97)	<RL of 1
Triclosan	µg/L	9 (3.3)	<RL of 0.2

[1]Average with std. dev. in ( ) (n=14). [2]<RL = result was less than the reporting limit based on the method used.

Fate of Pharmaceuticals and Hormones in Mounded Septic Drainfields.

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ABSTRACT

A variety of chemical compounds (known as emerging contaminants) are present in household wastewater due to the use and excretion of different products in the toilets, washers, kitchen, and sinks in the households. Many of these compounds are not completely removed by onsite wastewater treatment systems and can potentially contaminate groundwater. Our objective in this USDA-NIFA funded project was to investigate the occurrence, behavior, and leaching of select pharmaceuticals and hormones in septic system drainfields. Each drainfield received 3 L/ft^2/day of septic tank effluent (STE; equivalent to maximum allowable rate for Florida's sandy soils). Further, three small drainfields (1.5 m length x 0.9 m width x 0.9 m height) containing vertically stacked layers of soil (30 cm) and sand (30 cm) were constructed. Then, a drip line was placed and covered with 15-cm depth of sand and turf grass (St. Augustine) was planted to mimic a residential system. Below the drainfields, soil-water samples were collected using suction cup lysimeters and groundwater samples were collected using piezometers. Collected samples (STE, soil-water, groundwater, and leachate) were analyzed for four pharmaceuticals (acetaminophen, carbamazepine, ibuprofen, sulfamethoxazole) and three hormones (17β-estradiol, estrone, ethynylestradiol) by solid-phase extraction and liquid chromatography-tandem mass spectrometry (LC-MS/MS). In STE, ibuprofen, acetaminophen, and estrone were the most frequently (>80%) detected compounds. Among pharmaceuticals, ibuprofen was the most (>70%) frequently detected compound in soil-water and leachate. Concentrations of ibuprofen were highest in the STE (mean: 7,950 ng/L; n = 40), which reduced to <45 ng/L as STE percolated and leached from the drainfields. Concentrations of acetaminophen and estrone were 700 ng/L and 50 ng/L in STE, respectively. 17β-estradiol, was not detected in STE but was present in soil-water (32 ng/L) and leachate (25 ng/L), suggesting potential accumulation due to repeated applications of STE containing small amounts of 17β-estradiol. Ethynylestradiol was not detected in STE, soil-water, leachate, and groundwater samples. Our mass balance data shows that about 5-14% of applied compounds in STE were recovered in leachate (about 60 cm below drainfield), with the remainder (86-95%) either stored and/or degraded in the drainfield. We did not detect any compounds in groundwater (>700 cm below drainfields) after 7-months of STE dispersal. We *hypothesize* that some of the compounds are potentially mobile in the soil profile but may be further attenuated before STE reaches groundwater. After the drainfields are deconstructed in 2014, we will determine the amount of pharmaceuticals and hormones stored in the soil. Information from this study will be useful in determining the fate and transport of pharmaceuticals and hormones in septic system drainfields and their potential transport to groundwater in long-running septic systems.

Simulations of hydrologic effects on transport of surface-applied solutes and bacteria in a vadose zone-shallow groundwater continuum

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ABSTRACT

Better understanding of subsurface fate and transport of contaminants is vital to their proper monitoring and treatment. This study evaluated the effects of groundwater (GW) flow velocity on the transport of a miscible solute and bacteria in the capillary fringe (CF) and GW. Experiments were conducted using a glass-covered flow cell (90 cm-long × 50 cm-high × 3.5 cm-thick) packed with sand to visually evaluate the transport of a surface-applied red dye solution and green fluorescent protein-transformed *Escherichia coli* suspension in the vadose zone and GW under various simulated hydrologic conditions. Dye solution or *E. coli* suspension was applied to an area on the surface of the flow cell and their transports were monitored. Surface-applied solutes and bacteria were transported vertically in the unsaturated zone but horizontally when it reached the CF above the water table (WT) of a horizontally-flowing GW. At high GW flow velocity, horizontal transport of surface-applied contaminants were confined to the upper portion of the CF but moved deeper into it and eventually below the WT at slower velocity. These results suggest that there could be conditions wherein the collection of samples from the CF may be necessary when monitoring subsurface transport of surface-applied soluble chemicals and bacteria in locations with laterally-flowing shallow ground water.

Concerns over microbial and chemical contamination of GW and its subsequent effect on surface water quality highlight the need for an improved understanding of the fate and transport of microbes and solutes in the subsurface. Interest in subsurface fate of microbes is rooted from known cases of microbial contamination of both surface waters (USEPA, 2004) and subsurface drinking water sources (USEPA, 2006), and from the role that microbes play in the restoration of contaminated aquifers (Lee et al., 1988; Lovley, 1995; Scow and Hicks, 2005). Attention has been directed to chemical contaminants as they pose a hazard to the environment (Correll, 1998) while some may cause diseases in humans and animals (Gerba, 1996).

Various chemical and physical properties of both soil and contaminants affect the fates of chemical and microbial contaminants in the soil. Chemical pollutant properties such as net charge, essentiality to plants and microbes, and redox properties determine whether a given chemical is sorbed to the porous media, transformed, utilized by plants and microbes, or remains in solution for transport (Bedient et al., 1997). Microbial properties such as size, shape, hydrophobicity, and electrostatic charge determine whether a microbe is transported, sorbed, or strained/filtered in the soil (Ginn et al., 2002). In addition, the ability to compete for growth factors in the subsurface affects microbial survival (Coyne, 1999). When soluble chemical and microbial contaminants are applied to the soil in improper schemes and amounts, they can exceed the attenuation capacity of the soil and remain available for transport in the subsurface where their fate becomes largely determined by the hydrology of the system.

Subsurface contamination on a field or watershed scale (e.g., where a contaminant travels a long distance for an extended period of time) usually involves a plume that is tongue-shaped and deeper in the aquifer at the advancing front than at locations closer to the source (Frind and Hokkanen, 1987; LeBlanc, 1984). The dip in the plume front is largely attributed to years of recharge from precipitation that accumulate above the advancing plume, pushing it downward into the aquifer as it moves horizontally. At these scales, lateral transport of contaminants below the WT is the transport mechanism that primarily contributes to the overall movement of contaminants in the subsurface. In contrast, on a local scale (e.g., a distance of less than 25 m) transport between the contaminant source and a nearby location of interest; such as stream, ditch, drinking water well; could be accomplished in relatively short duration and before extensive precipitation is able to push the contaminants vertically into the aquifer effectively. In the latter case of transport scale, horizontal transport in the vadose zone, mainly within the CF, could be the major contributor to the overall subsurface horizontal transport of contaminants.

In general, surface-applied solutes have been described to move mainly vertically downward rather uniformly or in fingers within the upper part of the vadose zone with relatively low water content (Jawitz et al., 1998; Martin and Kroener, 1984). However, at regions in the vadose zone with high water content, such as the CF, the zone just above the WT that is saturated by capillary action, solute transport could shift from a predominantly vertical to a predominantly horizontal direction, as demonstrated in sand-packed flow cell experiments (Silliman et al., 2002) as well as field studies (Abit et al., 2008a, b). In addition, laboratory scale simulations have shown that up to 100% of surface-applied solutes can be transported horizontally in the CF without intersecting the WT (Amoozegar et al., 2006; Henry and Smith, 2002; Silliman et al., 2002). Horizontal transport in the CF was also demonstrated to appreciably contribute to the subsurface transport of solutes under field conditions (Abit at al., 2008a, b).

The impact of WT fluctuations on the retention of microbes and chemicals within the CF and the vadose zone above it have been recognized for decades (Stiles and Crowhurst, 1923; Abdul and Gillham, 1989; Jayatilaka and Gillham, 1996; Kao et al., 2001). Horizontal transport of bacteria in the CF has also been demonstrated in a laboratory-scale experiment where green fluorescent protein (GFP)-transformed *Escherichia coli* were transported via advection from below the WT to the CF (Dunn et al., 2004). This same experiment also showed that while in the CF, the microbes were transported horizontally and that coarse sand lenses above the WT served as the preferred paths for microbial transport.

Under current practices, the most widely used method of monitoring pollutant transport from waste disposal facilities and land application areas is ground water sampling at various depth intervals below the water table using sampling wells or piezometers installed at strategic locations (Fetter, 1999; Harter, 2003). In fact, Federal regulations [including the Resource Conservation and Recovery Act (RCRA); the Comprehensive Environmental Response, Compensation, and Liability Act (CERCLA); the Toxic Substance Act (TSCA); and the Federal Insecticide, Fungicide, and Rodenticide Act, (FIFRA)], as well as various state and local regulations only require collection and analysis of ground water samples for assessing ground water quality (Brown and Teplitzky, 1993). However, collecting only ground water samples for detecting pollutants may incorrectly assess the movement and fate of contaminants as the CF can impede vertical transport of pollutants into ground water. While it has been demonstrated that soluble solutes and microbes can be transported horizontally in the CF in the presence of horizontal GW flow, no study has been conducted evaluating the influence of hydrology on the extent of horizontal transport of contaminants in the CF. Moreover, because the WT is never static under field conditions, it is imperative to examine the fates of solutes and microbes in the CF as the WT fluctuates. This study was conducted to visually assess the effect of horizontal pore-water velocity on the horizontal transport of surface-applied solutes and bacteria in a CF-GW continuum.

MATERIALS AND METHODS

Experimental Set-up

Laboratory simulations of solute and bacterial transport were performed in a 90 cm-long, 50 cm-high and 3.5 cm-wide flow cell packed with medium sand (Fig. 1). The front side of the flow cell was covered with 0.64 cm-thick tempered glass that was not ultraviolet (UV) protected and the other sides were made of flat polyvinylchloride (PVC) sheets. Two 2.5 cm-wide chambers, with perforated inner walls were constructed on the two sides of the flow cell (see Fig. 1). The middle 85 cm of the flow cell (located between the two chambers with perforated walls) was evenly packed with medium sand material (effective diameter: 0.05 - 0.25 mm). The outlet chamber was connected to a section of Tygon plastic tubing with its open end fixed at 12 cm above the bottom of the flow cell. The inlet chamber was connected by Tygon tubing to a 25-L (Marriotte bottle) reservoir with the tip of the air inlet tube also set at 12 cm above the bottom of the flow cell. A simulated WT, at 12 cm above the bottom of the flow cell, was then established by introducing water (or broth solution in the case of the bacterial simulation) from the reservoir.

A desired slope of the WT that translated to a particular horizontal flow velocity across the flow cell was achieved by manipulating the elevation of the air inlet tube in the Marriotte bottle reservoir. Before any solute or microbial transport simulation was initiated, trial runs were conducted to determine the horizontal pore-water velocities as a function of the slopes of the WT for the experiment.

Dye Solution and Bacteria Suspension

For solute transport simulations, acid red dye (azophloxine) solution (0.5 g L^{-1}) was applied to the flow cell. For microbial transport simulations, *Escherichia coli* strain JM109 (Promega Corporation, Madison, WI) expressing green fluorescent protein (GFP) was used. Green fluoresces under UV light allowed visual observation of the *E. coli* cells. The *E. coli* was transformed with plasmid pGFPuv (Clontech Laboratories, Mountain View, CA.) using the standard transformation protocol (Promega, 2000). Transformed cells were grown in Luria Bertani (LB) broth (Difco Laboratories, Detroit, MI) with ampicillin (100 mg L^{-1}) at 37 ^{o}C and shaken at 100 revolutions min^{-1}. A 365 nm UV lamp (UVP, Upland, CA) was used to test the cultures periodically for fluorescence. When a visually intense fluorescence was observed, several glycerol stocks were made by adding 100 µL of culture to 400 µL of sterile glycerol and stored at -80 ^{o}C until use. Serial dilution plating determined that viable cell counts of the stocked culture were ~10^8 CFUs per mL. Thirty-six hours before a scheduled microbial transport simulation, the fluorescent bacterial suspension was mass-produced by inoculating 100 µL of glycerol stock into 250 ml Erlenmeyer flasks each with 125 mL sterile LB broth solution (with ampicillin) and incubated for 36 hours at 37^{o}C shaking at 100 revolutions min^{-1}.

Horizontal Transport of Dye and Bacteria

Evaluation of the horizontal transport of surface-applied dye solution and bacteria was conducted under three hydrologic conditions that included a flat WT and two different WT slopes. Specific WT slopes were achieved by elevating the air inlet tube in the Marriotte bottle reservoir by 1.3 cm (a 1.5% slope) or 2.6 cm (a 3% slope) from its 12-cm spot which marked a flat WT. Flow velocities were determined at the outlet and were found to be: 0 for the flat WT, approximately 82 cm d^{-1} for a WT slope of 1.5%, and approximately 160 cm d^{-1} for a WT slope of 3.0%.

Before each simulation, outlet discharge rates were monitored by four 1-hour trials to check whether constant water flow was achieved. During flat WT simulations, the inlet tube was disconnected from the Marriott bottle reservoir and the ends of both the outlet and inlet tubings were fixed 12 cm above the bottom of the flow cell. This allowed drainage through both ends of the flow cell when dye solution or bacteria suspension was added on the surface. The dye solution or bacteria suspension was added to the surface at a rate of 2.4 L d^{-1}. For the flat WT simulation, solutions were applied at application spot B (Fig. 1). For sloping WT simulations, solutions were applied at application spot A (Fig. 1). For solute transport, distilled water was used to establish the WT. For bacteria transport, half-strength LB-broth with ampicillin was used to establish the WT. After a simulation was started, time-lapse photographs were taken at 30-minute intervals for 10.5 hours. Bacterial transport simulations were conducted in a dark room and the flow cell was exposed to the 365 nm UV lamp only during the short instances when photographs were taken.

RESULTS AND DISCUSSION

Figure 2a shows the end condition of the simulation wherein the WT was flat (no horizontal water flow) when the dye solution was surface-applied. It was observed that the dye solution generally moved vertically downward in the unsaturated zone, and accumulated in the CF before it moved to below the WT. At the scale modeled, the bacterial suspension produced a similar plume behavior (Fig. 2b). Based on these observations, in the absence of horizontal groundwater flow, a solute or bacteria from a given source (e.g., a septic system drainfield or an animal waste retention pond) moving vertically through the vadose zone could be expected to initially accumulate in the CF and eventually move into the shallow groundwater provided that adequate volume of solution/suspension enters the system through the soil surface above the vadose zone.

When horizontal flow was induced by setting a 1.5 % slope of the simulated WT (pore-water velocity of ~82 cm d^{-1}), it was observed that the applied solute initially spread radially around the source and then generally moved vertically downward in the vadose zone as initially observed under no flow conditions (Fig. 3a and 3b). However, as soon as the solute plume reached the top of the CF, it showed indications of moving horizontally in the down-gradient direction (Fig. 3c). The plume was then transported horizontally towards the down-gradient direction within the CF without moving to below the WT (Fig 3d and 3e). These observations were consistent with the results of Amoozegar et al. (2006) and Silliman et al. (2002). Figure 3f shows that although a part of the plume eventually moved below the WT, most of it was transported horizontally within the CF.

When the horizontal GW gradient (i.e., slope of WT) was 3% (pore-water velocity of ~160 cm d^{-1}), the dye plume also moved generally downward with some radial dispersion while still above the CF (Fig. 4a and 4b) and was observed to be transported horizontally in the down-gradient direction as soon as it entered the CF (Fig. 4c and 4d). Compared to the condition when the GW pore-water velocity was ~82 cm d^{-1}, the horizontal part of the plume in the CF at GW pore-velocity of 160 cm d^{-1} tended to be thinner – flowing only at the upper portion of the CF (Fig. 4e and 4f). Moreover, 3.5 hours after the dye application, the plume fronts for the two flow rates were at approximately the same position (at approximately 35-40 cm from the inlet chamber) in the flow cell (Figs. 3c and 4c). However, after 7.5 hours, the front of the plume was already at the outlet when the pore-water velocity was ~160 cm d^{-1} (Fig. 4f). In contrast, at a pore-water velocity of only ~82 cm d^{-1}, the plume had only moved half the distance (~ 65 cm from the inlet chamber) indicating that the rate of movement of surface-applied solutes in the CF was proportional to the GW lateral flow velocity.

Transport of *E. coli* was also observed to follow the same general trend as the dye solute. When microbes were applied to one small area on the flow cell surface, they were initially transported downward in the unsaturated zone above the CF and horizontally in the down-gradient direction as soon as they reached the CF (Figs. 5 and 6). Moreover, as with the solutes, horizontal transport of microbes in the CF also tended to remain predominantly in the upper portion of the CF and seemed to be transported faster under higher horizontal GW flow rate.

The increase in pore-water velocity across the flow cell from ~82 cm d^{-1} to ~160 cm d^{-1} was achieved by increasing the rate of liquid application to the inlet chamber. The increase in rate of application to the inlet reduced the relative contribution of the surface-applied solution or suspension to the total amount of liquid that moved across the CF-GW continuum in the flow cell. This means that the dye or bacteria would tend to be transported in a thinner portion of the CF-GW continuum, and because they were surface-applied, it made sense that their horizontal transport would be isolated only at the upper part of the CF. In addition, horizontal transport through a relatively thinner portion (upper part) of the CF, as in the case with 3% WT slope, means that dye and bacteria moved across a transport path that had relatively smaller average cross-sectional area. Noting that the surface application rate remained constant across simulations, the reduction in the cross-sectional area of the transport path should result in faster rate of transport of the dye or bacteria as was observed in the flow cell with a higher GW pore-water velocity.

The above results and discussion could be summarized into four key points: 1) at the scale modeled, the general transport behavior of solutes and bacteria in a vadose zone and shallow GW continuum were comparable, 2) under relatively slow to no horizontal GW flow, surface-applied bacteria and solutes entered the GW, 3) at relatively high horizontal pore-water velocity, solute and bacterial transport became more isolated to the upper portion of the CF, and 4) the rate of solute transport in the CF increased with increasing horizontal GW velocity.

CONCLUSIONS

This study was conducted to simulate and evaluate the effect of horizontal pore-water velocity on the fate and transport of surface-applied solutes and bacteria in the capillary fringe (CF) and ground water (GW). Subsurface hydrology influenced the location where surface-applied solutes and bacteria were transported in the subsurface. When surface-applied solutes and bacteria reached the CF, they tended to be transported horizontally in the presence of horizontal GW flow. In addition to horizontal transport in the CF, contaminants were also transported horizontally below the WT at slower GW flow velocity. In contrast, horizontal transport of surface-applied contaminants tended to be isolated to the CF as horizontal GW flow velocity was increased. Confinement of contaminant transport to the upper portions of the CF at higher pore-water velocities also promoted the accelerated arrival of the plume at the outlet. Because solutes and bacteria in the CF can be transported horizontally in the presence of GW horizontal flow, the non-detection of these contaminants in GW samples does not necessarily indicate that surface-applied chemicals and bacteria cannot persist and be transported horizontally in the subsurface.

These observations can occur in scenarios involving onsite wastewater treatment systems. Examples would include cases wherein wastewater is surface-applied in areas with horizontally-flowing shallow groundwater or in sloping areas with conventional systems that have relatively thin vertical separation between the water table and the bottom of the trenches. Results from this experiment suggest that should there be a need to monitor subsurface horizontal transport of dissolved and bacterial contaminants in these areas, the protocol should include efforts to collect samples from the vadose zone, particularly the CF.

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Figures

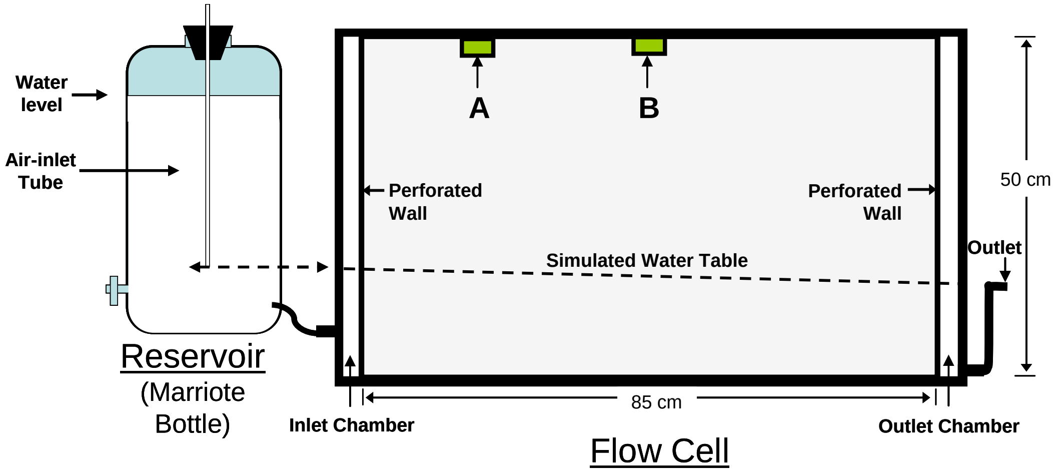

Figure 1. Two-dimensional illustration of the flow cell and the reservoir used in the visual modeling of solute and bacterial transport. The flow cell was 3.5 cm thick. Note: Illustration is not to scale.

Figure 2. Dye (a) and bacteria (b) plume in the flow cell in the absence of horizontal water flow after 8 hours of continuous surface application of dye solution or bacteria suspension. Photograph of bacteria plume was taken under 365 nm UV light and the green line is the location of the static water table.

Figure 3. Time-lapse photographs showing the area traveled by the surface-applied dye solution under a simulated water table (WT) slope of 1.5 %. The orange line represents the WT during simulated left-to-right ground water flow; the green line is the WT during static condition.

Figure 4. Time-lapse photographs showing the area traveled by the applied dye solution under a simulated water table (WT) slope of 3.0 %. The orange line represents the WT during simulated left-to-right ground water flow; and the green line represents the WT during static condition.

Figure 5. Time-lapse photographs under 365 nm UV light showing the area traveled by the surface-applied *E. coli* under a simulated water table (WT) slope of 1.5 %. The orange line represents the WT during simulated left-to-right ground water flow; and the green line represents the WT during static condition.

Figure 6. Time-lapse photographs under 365 nm UV light showing the area traveled by the surface-applied *E. coli* under a simulated water table (WT) slope of 3.0 %. The orange line represents the WT during simulated left-to-right ground water flow; and the green line represents the WT during static condition.

Characterization of Septic Tank Effluent from Coastal Residences

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ABSTRACT

Constituent concentrations in residential onsite wastewater treatment systems (OWTS) are important, among other things, as input parameters for nutrient loading models, total maximum daily load (TMDL) studies, ecosystem sustainability analyses, as well as for resource managers and regulatory decision-makers charged with setting and/or verifying compliance with established treatment standards. In these applications, if an actual wastewater concentration value is not readily available, a literature value (or an agreed upon assumed value) is used for critical calculations which may have a profound influence on outcomes, decision making, regulations, and management policies. To help provide additional data for decision-makers, an evaluation of septic tank effluent constituents, from residential households located in coastal communities in Rhode Island was undertaken. Septic tank effluent (STE) was collected from twelve full-time occupied homes and analyzed for biochemical oxygen demand (BOD_5), total suspended solids (TSS), fecal coliform bacteria, total nitrogen (TN), ammonium nitrogen (NH_4^+-N), nitrate nitrogen (NO_3^-N), total phosphorus (TP), chloride (Cl^-), pH, alkalinity, dissolved oxygen (DO), and temperature. In addition, actual hydraulic flow data for each system was collected.

Hydraulic flows for the study systems varied from 212 to 1,241 L/d (56 to 328 gpd) per household, representing a per capita flow from 106 to 248 L/d (28 to 66 gpd). An analysis of the trimmed data (where the lower 10% and upper 10% of the raw data values were excluded to remove extremes) yielded: mean BOD_5 concentrations in STE ranged from 145 to 368 mg/L; mean TSS concentrations ranged from 39 to 76 mg/L; mean fecal coliform bacteria concentrations ranged from 7.60E+04 to 1.17E+07 cfu /100mL; mean TP concentrations ranged from 8 to 13 mg/L; mean TN concentrations ranged from 34 to 104 mg/L, with most of the TN in the NH_4^+ form (mean ranged from 32 to 95 mg/L). Low hydraulic flow per person in one study system yielded elevated concentrations for all constituents analyzed, illustrating the potential influence of carriage water volume on wastewater constituent concentrations.

Introduction

State regulatory agencies need STE constituent values to evaluate the treatment performance of emerging OWTS technologies. Established regulatory jurisdiction N removal treatment standards (or guidelines) in the northeast U.S. use 19 to 25 mg/L as the TN final effluent concentration and a minimum of 50 to 55% TN reduction from STE values as treatment thresholds (RIDEM, 2012; MADEP, 2014). This infers that the STE could be as low 38 mg/L TN, a concentration typical of influent at municipal sewage treatment plants (Tchobanoglous and Burton, 1991). An issue confronting regulatory managers of nitrogen-sensitive coastal water resources is that verification of the fraction of TN removed in some advanced onsite wastewater treatment technologies utilizing a single processing tank is daunting, because the influent raw wastewater is altered by mixing with recirculated, lower nitrogen flow. In these instances decision-makers may use in-house STE data values gathered from sampling other septic tanks in their area; however, these data usually does not exist and they would typically use published literature values.

The range in literature values for STE constituents varies widely (Table 1), leading Crites and Tchobanoglous (1998) to conclude that there is no such thing as "typical wastewater" and that data labeled as such should be used only as a general guide. To help define wastewater parameters, the Consortium of Institutes for Decentralized Wastewater Treatment (CIDWT) defined residential STE as having less than 170 mg BOD_5/L; less than 60 mg TSS/L; and, less than 25 mg fats, oils and grease (FOG)/L. Values exceeding these constitute high strength wastewater (CIDWT, 2009). Although it is useful to define the threshold between residential and high strength wastewater, this definition focuses on a limited number of parameters, and does not include critical nutrient and bacterial constituents, indicating that further study is needed.

MATERIALS AND METHODS

To help provide additional STE composition data for decision-makers in Rhode Island and elsewhere, wastewater was collected from the outlet of 12 septic tanks that served as the primary treatment components for innovative and alternative technology treatment trains installed to serve single-family homes in Rhode Island coastal communities. All septic tanks were new, code compliant watertight concrete tanks sized as either 3785 or 5678 liter (1,000 or 1,500 gallon) for a 3 and 4 bedroom home design, respectively. These systems were installed under the auspices of state and federally-funded OWTS demonstration projects evaluating pathogen and N removal technologies. Four of the systems (Table 2; Site IDs GH – 3M, 4H, 5M and 6S) were sampled 35 times over a 5.5-year period, from September 1999 through January 2005. System GH 4H Current was sampled 49 times from January 2013 to January 2014 and will continue to be sampled as part of an on-going climate change research project (these proceedings; Cooper et al., 2014). Systems GH 4H and GH 4H Current are the same system, receiving wastewater from the same family; however, one less adult occupant now lives in the home. The other 8 systems (with the AF site designations) were sampled 15 times over a 1.5-year period, from August 1997 through March 1999.

All sampling was conducted by staff of the New England Onsite Wastewater Training Center following approved field sampling quality assurance project plan procedures. Laboratory analyses included BOD_5, TSS, fecal coliform bacteria, TN, NH_4^+-N, NO_3^--N, TP, Cl^-, pH, and alkalinity. Dissolved oxygen (DO), temperature and actual hydraulic flow through the systems were determined in the field (all systems had water meters installed on pressurized drainfield lines). Laboratory analyses on earlier (GH and AF identified) systems were conducted at the University of Rhode Island (URI) Watershed Watch Laboratory; whereas, Site GH 4H Current system sampling and analyses were conducted by the URI Laboratory of Soil Ecology and Microbiology. All analyses followed standard methods and procedures (APHA, 1998). Details of these systems are outlined in prior published papers (Loomis et al., 2001; 2004) and in these proceedings (Cooper et al., 2014).

RESULTS AND DISCUSSION

Our results are reported in two ways: (1) trimmed data for each site, where the lower 10% and upper 10% of the raw data values were excluded to remove extremes (Table 2), and (2) a pooled (or composite) summary of the trimmed data for the systems to yield a single value for each constituent (Table 3). Trimming the lower and upper 10% of the raw data, combined with a fairly large number of observations, produced single mean and median concentration values that were quite similar, and resulted in tighter standard deviations (s.d.) around the mean for each constituent in the pooled data set.

Fecal coliform bacteria, BOD_5, and TSS

A fair amount of variability in concentrations of STE constituents occurred within an individual system as well as among the twelve study sites (Table 2). This was most pronounced for fecal coliform bacteria, where means ranged from 7.60E+04 to 1.17E+07 colony-forming units (cfu)/100mL and coefficients of variation (CV) ranged from 1 to 1.8 (CV calculations not included in Table 2). Mean BOD_5 concentrations in STE ranged from 145 to 368 mg/L, and TSS concentrations ranged from 39 to 76 mg/L (Table 2). The mean and the range of fecal coliform, TSS, and BOD_5 concentrations for the systems in our study were within the range of those reported by Oregon DEQ (2006); however, BOD_5 concentrations for six of the twelve systems in our study exceeded those values reported by others (referenced in Table 1) and CIDWT (2009).

System GH 5M had the highest fecal coliform bacteria and BOD_5 values of all the systems, yet it had the lowest volume of carriage water per person generated of all the residences (Table 2). This reflects the influence of low hydraulic flow on elevating wastewater strength. Intuitively, low carriage water usage in a home would likely produce lower TSS concentrations in STE, as lower flows would promote longer hydraulic retention times in the septic tank, resulting in better solids settling and enhanced TSS removal. The lower flows in system GH 5M did not produce the lowest TSS concentrations observed, but the value was in the lower third of the systems with TSS values $\leq$ 43 mg/L. Pooled trimmed data means for BOD, TSS and fecal coliform bacteria for all the study systems were 240, 49 mg/L, and 9.54E+05 cfu/100mL, respectively (Table 3). The pooled data summary yielded CVs for BOD and TSS that were reasonably low (0.23 and 0.36, respectively), whereas the CV for fecal coliform bacteria was 3.35, illustrating the high variability in bacteria data. Despite this variability, these values remain in general agreement with studies referenced in Table 1, as well as those reported by Oregon DEQ (2006), and CIDWT (2009).

Nutrients

Septic tank effluent mean TN concentrations for the systems ranged from 34 to 104 mg/L, with the N present primarily as NH_4^+, and less than 0.5 mg/L in the NO_3^--N form (Table 2). Mean TP concentrations ranged from 8 to 13 mg/L for the systems. Maximum, minimum, and standard deviation values displayed some variability for each system, but are similar to those reported by others (Table 1; Oregon DEQ, 2006). The highest TN and TP values were for system GH 5M, which had low carriage water generation. The pooled trimmed data yielded mean + s.d. TN and TP concentrations of 62 $\pm$13 and 10 $\pm$ 2 mg/L, respectively, and CV $\leq$ 0.21 (Table 3). The pooled TN mean value in our study is similar to that found by others, e.g. 64 mg/L (Lowe et al., 2009), 63 mg/L (Oregon DEQ, 2006), and 62 mg/L (Converse, 2004).

Cl^-, pH, Alkalinity, and D.O.

As noted in Table 2, chloride, pH, alkalinity and DO data are available only for systems GH – 3M, 4H, 4 H Current, 5M and 6S. Mean Cl^- concentrations for all of the GH systems ranged from 48 to 97 mg/L; mean pH values ranged from 6.4 to 7.4; and mean alkalinity (as $CaCO_3$) ranged between 262 and 428 mg/L (Table 2). The GH pooled trimmed data for Cl^- and alkalinity yielded means of 62 and 316 mg/L, respectively; whereas pH was 7.0 (Table 3). These pooled Cl^- values are well within the ranges reported by Anderson et al. (1994) and Crites and Tchobanoglous (1998). Dissolved oxygen concentrations in STE for these sites were all below 0.3 mg/L, indicating that anaerobic conditions existed in all the septic tanks.

Sites GH 4H and 6S both had municipal water service, and thus had a higher source alkalinity than commonly seen in private drinking water wells located inland from the Rhode Island coastline. Sites GH 3M and 5M utilized shallow wells for water supply, yet had comparable (and for site 5M, much higher) alkalinity in their STE than the two homes served by municipal water (sites 4H and 6S). Oregon DEQ (2006) reported similar STE alkalinity levels in their Deschutes County study, but soil and groundwater alkalinity levels in Oregon are likely to be much higher than those found under the acidic soil conditions common in Rhode Island.

Alkalinity levels in STE from system GH 5M may be somewhat influenced by the low carriage water generation mentioned above, but the potential of saltwater intrusion into the (near shoreline) home drinking water well at this site may have helped to elevate alkalinity. System 5M also had the highest concentrations of Cl^-, lending support to the saltwater intrusion hypothesis.

Temperature and Wastewater Flow

Because of seasonal variation in ambient air temperature, we analyzed STE temperature for the four systems noted in Table 2 (GH – 3M, 4H, 5M and 6S) in three ways: cold season (the mean of all November through April readings), warm season (all May through October readings), and overall temperature (all temperature data without regard to season). STE temperatures for these systems were reflective of season, with cold season temperatures being 4 to 5^oC colder than warm season (Table 2). System GH 5M had the lowest STE temperatures (Table 2) of all the GH systems and for any season; this is perhaps a function of lower volumes of warm carriage water entering the septic tank and thus greater susceptibility to changes in ambient air temperature. The pooled data mean STE temperature was 15^oC (overall), 11^oC (cold season), and 19^oC (warm season; Table 3).

Hydraulic flows for the 12 individual study systems varied from 212 to 1,241 L/d per household (56 and 328 gpd), representing a per capita flow of 106 to 248 L/d (28 to 66 gpd; Table 2). It was only possible to calculate the pooled mean flow for the five GH site ID systems noted in Table 2. The pooled mean per capita flow for these five GH systems was 195 L/d (52 gpd) (Table 3), which is similar to the 49.7 gpd reported by Converse (2004).

The lowest flow generated by the 12 homes was observed at site GH 5M, where the residents practiced strict water conservation measures. The low carriage water volume generated in system 5M produced higher concentrations of all constituents (Tables 2 and 3), a finding that we reported about in an earlier study, and resulted in this system not meeting the Rhode Island TN treatment standards (Loomis et al., 2004). This issue has prompted Rhode Island regulators to consider changes to the N removal standards that also include TN loading in addition to the existing concentration and percent reduction criteria.

The residence for system AF W-VE produced the highest household (1,241 L/d; 328 gpd) and per capita wastewater flows (248 L/d; 66 gpd) of all the systems (Table 2). When compared to the lowest carriage water system (GH 5M) and the pooled values for all other systems studied (Table 3 pooled data), it appears that this system produced lower constituents concentrations, suggesting the dilution influence of higher carriage water volumes on wastewater strength.

The amount of TP generated per capita at our study sites ranged from 0.4 to 0.8 kg/yr (0.9 to 1.8 lbs/yr), and the per person TN generation ranged from 2.1 to 5.6 kg/yr (4.6 to 12.3 lbs/yr). These values agree with those reported by USEPA (2002). The TN generated per capita at the two sites exhibiting the greatest extremes in flow (highest flow site AF W-VE, and lowest flow site GH 5M) was the same at 4.0 kg/yr. However, the potential TN loading from system AF W-VE was 20.4 kg/yr, compared to 8.0 kg/yr for system GH 5M. This underscores the importance of using loading as an essential parameter when evaluating a system's compliance with treatment standards in N sensitive watersheds.

System GH 4H comparisons

Systems GH 4H and GH 4H Current are the same system, receiving wastewater from the same family. System GH 4H was monitored for 5.5 years ending in January 2005, through which time 3 adults occupied the home on a full time basis. System GH 4H Current has been monitored on a weekly basis since January 2013, and now receives wastewater from 1 adult full time and another with partial occupancy (work-related travel). As a result, household flow to this system was reduced from 583 L/d (154 gpd) to 295 L/d (78 gpd). This flow reduction appears to have produced a marked increase in BOD_5 concentrations (from 198 to 245 mg/L), and fecal coliform bacteria, TN, and Cl^- all trended slightly upward. TP concentrations also increased sharply from 8 to 12 mg/L, representing a 33% increase in concentration, and TSS showed an 11% decrease. These findings suggest that the concentrations of particular constituents (notably BOD_5 and TP) are likely to respond to changes in a household's wastewater flow, whereas others are not similarly influenced.

CONCLUSIONS

We investigated the concentrations of constituents in effluent from septic tanks serving 12 coastal residences in Rhode Island, as well as hydraulic flow. Our study confirmed, as others have, that variations in STE exist for a particular home, as well as among households. Trimming data to eliminate 10% of the extremes on either end of our raw data set helped to minimize the variation expressed in each system. Pooling the trimmed data for all 12 systems resulted in a larger number of observations for a particular parameter, and yielded more stability in means, closer agreement between median and mean values, lower standard deviation values, and less scatter in the range of values. Using this pooled data gave us a single mean and associated standard deviation useful for comparing our results to those reported by others.

Our pooled trimmed data means were 9.54E+05 cfu/100mL for fecal coliform bacteria; 240 mg/L for BOD_5; 49 mg/L for TSS; 62 mg/L for TN; and 10 mg/l for TP. Pooled per capita flow in our study was 195 L/d (52 gpd). These values will be useful for regulatory agencies in Rhode Island and other states in evaluating, developing and refining policies related to onsite wastewater management, nitrogen removal technologies, high strength wastewater criteria, and TMDL studies. Important consideration should be given to the variability of these numbers.

Because of the long-term nature of our study, we were able to monitor the STE from one home (GH 4H) occupied by the same individuals for 15 years. During that period, a reduction in occupants had occurred allowing us to evaluate the influence of occupancy on wastewater constituent concentrations. Constituent analysis results from this system suggests that, providing that the per capita flow still remains the same, BOD_5 and TP concentrations may be more influenced by a reduction in home occupancy than would other wastewater constituents.

Low carriage water generated from the GH 5M study site (212 L/d; 56 gpd) produced the highest constituent concentrations of all the sites, and in particular TN (104 mg/L). In a previous N treatment technology study this same system experienced difficulty meeting Rhode Island TN standards. In some cases, low carriage water flow systems may produce less TN loading per person (kg/yr) and have less potential impact on receiving waters than neighboring systems that meet treatment concentration standards, but generate much higher wastewater flows. Conversely, high carriage water generation may have a dilution effect on constituent concentrations, enabling a system to meet a concentration threshold, yet still have high nutrient loading. Regulatory programs need to recognize these issues, and develop treatment standards and policy measures that include a loading parameter, and do not penalize homeowners who adhere to good water conservation practices, yet exceed final effluent concentrations.

ACKNOWLEDGEMENTS

The authors thank the following funding agencies that provided past support to monitor these systems: USEPA – NODP Phase II and National Community Decentralized Wastewater Treatment Project; Rhode Island Department of Environmental Management; and, past and current funding from the Rhode Island Agricultural Experiment Station. This document is produced under the auspices of the Rhode Island AES, USDA Hatch Multi-state NE-1045 Project.

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Table 1. Mean (range) wastewater constituent concentrations in septic tank effluent as reported in published literature.

Constituent	**Anderson, et al. (1994)**	**Converse (2004)**	**USEPA (2002)†**	**Crites and Tchobanoglous (1998)**	**Kaplan (1991)**
BOD_5 (mg/L)	93.5 (46-156)	173 (6-305)	140-200	150-250	138 (64-256)
TSS (mg/L)	‡NR	85 (13-207)	50-100	40-140	155 (43-485)
Fecal coliform bacteria (cfu/100mL)	$10^3 – 10^5$	2.80E+06 (3.51E+03-1.10E+08)	$10^6 – 10^8$	$10^3 – 10^7$	NR
TN (mg/L)	44.24 (19-53)	61.7 (37-199)	40-100	50-90	45 (9-125)
TP (mg/L)	8.6 (7.2-17)	NR	5-15	12-20	NR

†Source: Van Cuyk and Siegrist, 2001. ‡NR = not reported

Table 2. Trimmed† wastewater constituent concentrations in septic tank effluent from coastal residences.

Site ID	**BOD$_5$ Mean S.D. (range)** mg/L	**TSS Mean S.D. (range)** mg/L	**Fecal coliform bacteria Mean S.D. (range)** cfu/100mL	**TN Mean S.D. (range)** mg/L	**NH_4-N Mean S.D. (range)** mg/L	**NO_3-N Mean S.D. (range)** mg/L	**TP Mean S.D. (range)** mg/L	**Cl^- Mean S.D. (range)** mg/L	**pH Mean S.D. (range)** std. units	**Alkalinity Mean S.D. (range)** mg/L	**D.O.§ Mean S.D. (range)** mg/L	**Temp.§ Means Overall Cold Warm** ^oC	**Flow Total & Per Capita Means** L/d(gpd)
GH 3 M	272 37 (202-353) n=28	39 14 (21-63) n=26	5.22E+05 8.96E+05 (9.00E+03-3.80E+06) n=28	63 8 (53-78) n=24	56 11 (41-73) n=17	0.15 0.09 (0.05-0.20) n=3	11 2 (8-14) n=15	76 22 (49-124) n=22	6.8 0.2 (6.5-7.1) n=27	262 42 (119-318) n=19	0.20 0.29 (0.01-1.7) n=31	16 13 18 n=31	587(155) 196(52) n=30
GH 4 H	198 33 (128-249) n=29	45 13 (22-67) n=27	2.73E+05 3.72E+05 (2.00E+04-1.36E+06) n=28	69 6 (57-79) n=29	64 9 (45-74) n=17	0.06 -- (0.06) n=1	8 1 (7-9) n=13	48 4 (41-54) n=22	7.1 0.2 (6.7-7.4) n=28	288 36 (233-360) n=20	0.20 0.30 (0.01-1.90) n=33	15 12 17 n=33	583(154) 193(51) n=28
GH 4 H Cur-rent	245 53 (160-340) n=40	40 6 (28-50) n=27	6.48E+05 6.81E+05 (1.20E+05-2.60E+06) n=36	71 6 (62-82) n=40	55 6 (44-66) n=37	0.15 0.13 (0-0.41) n=38	12 2 (9-14) n=23	51 6 (37-60) n=20	6.4 0.2 (6.2-6.7) n=34	‡NA	0.10 0.10 (0-0.25) n=20	15 NA NA n=32	295(78) 182(48) n=8
GH 5 M	358 74 (255-533) n=24	43 16 (23-88) n=27	1.17E+07 1.52E+07 (3.00E+05-6.00E+07) n=28	104 9 (91-124) n=29	95 6 (86-108) n=17	0.11 0.05 (0.05-0.20) n=9	13 2 (11-16) n=12	97 6 (87-108) n=21	7.1 0.2 (6.8-7.6) n=28	428 50 (317-515) n=19	0.14 0.07 (0.04-0.33) n=26	14 10 15 n=26	212(56) 106(28) n=26
GH 6 S	218 28 (167-271) n=29	49 16 (33-81) n=27	1.46E+06 1.38E+06 (9.00E+04-5.50E+06) n=28	65 6 (55-76) n=29	67 8 (47-78) n=17	0.14 0.08 (0.05-0.28) n=7	10 1 (8-12) n=15	48 3 (44-56) n=22	7.4 0.2 (7.1-7.7) n=28	303 34 (261-359) n=20	0.15 0.10 (0.02-0.54) n=32	16 13 17 n=32	476(126) 238(63) n=30

†Trimmed data = 80% of the raw data, where the upper 10% and the lower 10% of values have been excluded to remove extremes. §Raw data used for D.O. and temperature for all GH systems. ‡NA= not analyzed

Table 2. Trimmed† wastewater constituent concentrations in septic tank effluent from coastal residences. (continued)

Site ID	**BOD Mean S.D. (range)** mg/L	**TSS Mean S.D. (range)** mg/L	**Fecal coliform bacteria Mean S.D. (range)** cfu/100mL	**TN Mean S.D. (range)** mg/L	**NH_4-N Mean S.D. (range)** mg/L	**NO_3-N Mean S.D. (range)** mg/L	**TP Mean S.D. (range)** mg/L	**Cl^- Mean S.D. (range)** mg/L	**pH Mean S.D. (range)** std. units	**Alkalinity Mean S.D. (range)** mg/L	**D.O. Mean S.D. (range)** mg/L	**Temp. Means Overall Cold Warm** °C	**Flow Total & Per Capita Means** L/d(gpd)
AF N-LI	300 34 (225-342) n=11	42 18 (24-93) n=11	2.36E+05 2.48E+05 (5.90E+03-7.10E+05) n=7	49 8 (30-56) n=9	47 6 (39-62) n=11	0.16 0.03 (0.12-0.19) n=5	10 2 (7-13) n=11	‡NA	NA	NA	NA	NA	643(170) 128(34)
AF N-LO	145 50 (90-243) n=12	57 20 (32-90) n=11	3.38E+05 3.75E+05 (3.80E+04-1.00E+06) N=6	51 12 (29-65) n=12	56 6 (46-63) n=12	NA	9 2 (7-13) n=11	NA	NA	NA	NA	NA	503(133) 168(44)
AF N-WR	202 51 (144-273) n=5	57 15 (36-83) n=9	6.09E+05 2.07E+05 (3.09E+05-9.50E+05) N=8	44 9 (30-56) n=13	43 9 (26-58) n=13	NA	9 1 (7-11) n=11	NA	NA	NA	NA	NA	515(136) 129(34)
AF P-DU	368 62 (304-467) n=5	58 18 (39-84) n=9	1.07E+06 8.85E+05 (1.80E+05-2.90E+06) n=9	41 7 (31-52) n=12	46 5 (36-52) n=13	0.23 0.04 (0.19-0.27) n=3	8 1 (6-11) n=13	NA	NA	NA	NA	NA	1022(270) 255(68)
AF P-RH	183 82 (48-349) n=9	49 28 (27-112) n=8	1.33E+05 1.63E+05 (6.00E+03-4.50E+05) N=6	56 10 (38-69) n=13	58 7 (46-68) n=13	NA	10 3 (6-17) n=11	NA	NA	NA	NA	NA	424(112) 106(28)

†Trimmed data = 80% of the raw data; where the upper 10% and the lower 10% of values have been excluded to remove extremes. ‡NA= not analyzed

Table 2. Trimmed† wastewater constituent concentrations in septic tank effluent from coastal residences. (continued)

Site ID	BOD Mean S.D. (range) mg/L	TSS Mean S.D. (range) mg/L	Fecal coliform bacteria Mean S.D. (range) cfu/100mL	TN Mean S.D. (range) mg/L	NH_4-N Mean S.D. (range) mg/L	NO_3-N Mean S.D. (range) mg/L	TP Mean S.D. (range) mg/L	Cl^- Mean S.D. (range) mg/L	pH Mean S.D. (range) std. units	Alkalinity Mean S.D. (range) mg/L	D.O. Mean S.D. (range) mg/L	Temp. Means Overall Cold Warm °C	Flow Total & Per Capita Means L/d(gpd)
AF W-LI	193 37 (139-272) n=13	67 18 (40-92) n=9	1.39E+05 1.39E+05 (1.30E+04-3.60E+05) n=8	34 6 (21-42) n=12	32 5 (21-44) n=13	0.45 0.15 (0.12-0.40) n=3	11 3 (6-15) n=12	‡NA	NA	NA	NA	NA	893(236) 179(47)
AF W-MC	199 49 (119-258) n=12	51 10 (43-71) n=9	1.82E+05 2.13E+05 (3.20E+04-5.30E+05) n=8	42 6 (28-51) n=12	40 4 (31-46) n=12	NA	9 2 (7-13) n=11	NA	NA	NA	NA	NA	984(260) 197(52)
AF W-VE	190 118 (118-298) n=8	76 34 (47-152) n=8	7.60E+04 5.28E+04 (1.40E+04-3.40E+05) n=9	45 11 (24-61) n=12	43 11 (19-56) n=12	0.34 0.23 (0.18-0.61) n=3	9 1 (7-11) n=12	NA	NA	NA	NA	NA	1,241 (328) 248(66)

†Trimmed data = 80% of the raw data; where the upper 10% and the lower 10% of values have been excluded to remove extremes. ‡NA= not analyzed

Table 3. Pooled trimmed data† for residential septic tank effluent from coastal communities.

Parameter	Mean	Median	S. D.	Coefficient of variation (CV)	Min	Max	Count (n)
Fecal coliform bacteria (all systems) (cfu/100mL)	9.54E+05	4.10E+05	1.37E+06	3.35	3.20E+04	7.90E+06	172
BOD (all systems) (mg/L)	240	233	53	0.23	151	360	214
TSS (all systems) (mg/L)	49	45	16	0.36	25	92	208
TN (all systems) (mg/L)	62	63	13	0.21	36	95	237
NH_4-N (all systems) (mg/L)	54	54	11	0.20	35	78	192
NO_3-N (all systems) (mg/L)	0.17	0.17	0.10	0.60	0.0	0.41	70
TP (all systems) (mg/L)	10	9	2	0.19	7	14	168
Cl^- (GH only) (mg/L)	62	53	18	0.35	44	105	102
pH (GH only) (std. units)	7.0	7.0	0.31	0.04	6.4	7.5	138
Alkalinity (GH; except 4H Current) (mg/L)	316	297	60	0.20	233	457	79
DO (GH only) (mg/L)	0.13	0.13	0.06	0.42	0.02	0.25	113
Temp overall (all dates combined) **(GH only)** (°C)	15	15	3.5	0.23	9	20	137
Temp cold season (GH; except 4H Current) (°C)	11	11	2.3	0.21	6	16	59
Temp warm season (GH; except 4H Current) (°C)	19	19	1.5	0.08	15	21	53
Flow per home (GH only) (L/d) (gpd)	462 122	492 130	140 37	0.28	204 54	679 179	119
Flow per capita (GH only) (L/d) (gpd)	195 52	212 56	60 16	0.28	86 23	291 77	119

† Trimmed data = 80% of the raw data; where the upper 10% and the lower 10% of values have been excluded to remove extremes.

Determining Measurement Range and Other Important Technical Specifications for Aardvark Permeameter.

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ABSTRACT

Aardvark Permeameter is a constant-head borehole permeameter. Aardvark does not use a Marriott Bubble Tower for establishing a constant head height at the bottom of borehole. Instead, it uses float valve technology. Float valve technology eliminates “bubble noise” (which is a common issue related to using Marriott Bubble Tower technology) and therefore it is expected to add to measurement accuracy. However, in soils with extremely low permeability, small amounts of leaking (from float valve), or even evaporation may distort measurement results.

Considering significant differences between Aardvark methodology and permeameters based on Marriott Bubbler, there is a need for determining important technical specifications (e.g. measurement range, suitable reading intervals, etc) for Aardvark.

Over head pressure has a significant impact on Aardvark water supply (from reservoir into borehole). Aardvark water supply rate was measured under minimum practical overhead pressure (7 kPa or 1 psi) and maximum nominal applicable pressure (34 kPa or 5 psi). Results were used for determining Aardvark operational rage.

It is not possible to determine Aardvark leaking (from float valve) rate in soil. The reason is that there is no way for distinguishing between valve leakage and soil percolation rate. Therefore Aardvark system was installed in laboratory and in a clear pipe with 10 cm (4 inch) diameter (recommended borehole diameter for Aardvark). The pipe was completely impermeable to water. Therefore it was possible to detect very small amounts of water loss (due to leakage or evaporation) from Aardvark system. Evaporation from borehole was measured using another identical cylinder and Aardvark measurements were corrected for evaporation.

Overhead pressure above Aardvark Permeameter Unit was adjusted on 41 kPa (6 psi). It was 7 kPa (1 psi) more than maximum nominal pressure (34 kPa or 5 psi) for Aardvark to get reliable results. Aardvark software application was used to perform automated readings (every minute) for more than 20 hr. Reading procedure was repeated 15 times. Reading data was used for simulating leakage rate and evaporation rate at any time increment.

A mathematical model was created to calculate Saturated Hydraulic Conductivity (K) based on four methods of calculation and due to different scenarios. Calculation methods include Glover solution, Reynolds and Elrick solution, Radcliff and West method and Earth Manual method. For each level (order of magnitude) of K, maximum and minimum of Flow Rate were calculated. Measurement Error was calculated using Aardvark Scale accuracy and resolution and also system water loss due to leakage and evaporation.

A mathematical model was created for simulating Aardvark leakage rate at any time increment (1 minute) after opening Aardvark Reservoir valve. Since leakage Error reduces over time, Reading Start Time was defined as time duration between opening Aardvark Reservoir valve and starting "reliable" (less than 5% Measurement Error) readings (due to small Leakage Errors). Using Flow Rate, Leakage Rate and Measurement Error (less than 5%), Reading Intervals and Reading Start Time were optimized for each level of K.

Aardvark practical operational range is $10^{-4} > K > 10^{-9}$ m/s. A 1-min Reading Interval is well enough for soils with $K > 10^{-6}$ m/s. Reading Interval in the range of 10^{-7} m/s is 1 to 15 min. Longer Reading Intervals (15 min to 8 hr) are needed for soils in range of 10^{-8} m/s. Reading Intervals for K values in range of 10^{-9} m/s are very long (8 to 24 hr). However, using Aardvark automated reading feature, it is still possible to measure these soils.

Reading start time for soils with K values bigger than 10^{-5} m/s is 5 min. For soils in range of 10^{-6} Start Time is between 5 to 10 min. For soils in range of 10^{-7}, Start Time is between 10 to 35 min. Start time for soils in range of 10^{-8} m/s is 35 min to 4 hr. In extremely slow soils (K value in range of 10^{-9} m/s) Start Time can vary from 4 to 48 hr.

Aardvark was capable of performing reliable and repeatable measurements automatically and for long periods of time which makes it a suitable instrument for measuring saturated hydraulic conductivity in laboratory and field condition.

Development of a GIS based decision support toolset to assess the feasibility of on-site wastewater treatment and disposal options in low permeability subsoils

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ABSTRACT

Traditional on-site wastewater treatment systems have proven to be unsuitable in areas of low permeability subsoils representing a risk to human health and the environment. With large areas being covered by low permeability tills, Ireland needs to consider alternative treatment and disposal options to be able to allow further development in these areas and to deal with polluting legacy sites. The paper describes the development and structure of a GIS based decision support toolset to evaluate possible alternative strategies for these sites. The programme takes as its initial input the proposed site of a new dwelling or the location of an existing house located in an area of low permeability subsoils. Through a series of interconnected GIS geoprocesses the model outputs appropriate solutions for a site ranking them in terms of environmental sustainability and cost. However, the final decisions are still dependent on on-site constraints so that each solution is accompanied by an alert message that provides additional information for the user to refine the output list according to the available local site specific information.

INTRODUCTION

The domestic wastewater of over one third of the population in Ireland is treated by on-site wastewater treatment systems (OSWWTSs). For single houses in areas with no main drainage on-site systems typically consist of septic tanks followed by a percolation area (soil attenuation system) (Gill et al., 2007). However, where the subsoil permeability is not sufficient to take the effluent load, surface ponding and runoff of pollutants to surface waters may occur. This represents a serious health risk and can also contribute to eutrophication in nutrient sensitive water bodies. Hence, a lower limit on subsoil permeability was defined by the Irish Environmental Protection Agency below which, at typical on-site wastewater hydraulic loads, percolation into the ground will not be fast enough and therefore discharge to ground is not permitted (EPA, 2009). The proportion of the country with inadequate percolation is estimated at 39% (EPA, 2013) and according to the current legislation further house development in such areas would probably be very limited. Furthermore, existing houses in such areas may represent both a risk to human health and nearby surface waters.

Therefore the aim of this research is to investigate alternative wastewater treatment and disposal options for rural housing in these areas and to develop a web based GIS (Geographic Information Systems) decision support toolset for Local Authority planners and managers to evaluate alternative strategies on the basis of both cost-benefit and environmental sustainability principles.

MATERIAL AND METHODS

Geospatial modelling was conducted through the use of ESRI's ArcGIS 10 to evaluate the alternative strategies. Houses located outside of mapped sewered urban and rural areas were assumed to use a septic tank system for the treatment of their wastewater. To identify legacy septic tanks that are situated in areas of low subsoil permeability and therefore potentially present the highest risk of surface water pollution, a map specifying the likelihood of inadequate percolation for OSWWTSs in Ireland was used which combines the relationship between groundwater vulnerability, recharge, soil/subsoil permeability and surface runoff (EPA, 2013). This map was used to clip the GIS layer of assumed septic tanks and to extract the site locations of interest. The alternative on-site disposal systems which were considered as possible solutions at these problematic sites were pressurised distribution systems, i.e. low pressure pipe (LPP) or drip distribution (DD) systems (EPRI, 2004; USEPA, 1999), sealed basin evapotranspiration systems (Arias, 2012; Curneen and Gill, 2014; Gregersen and Brix, 2001) as well as closed collection tanks (cesspools) with regular emptying (Norström et al., 2008) and disposal at centralised wastewater treatment plants. Where the impermeable soil layer is shallow enough, the discharge of treated effluent through an imported media filter into more permeable subsoil or bedrock was also considered. Furthermore, the possibility to connect houses to the nearest existing sewer network or the feasibility of clustering together several houses that could be served by a decentralised treatment plant with a consented discharge to a water course were assessed.

A modelling architecture (decision matrix) was developed for the various scenarios, incorporating the use of geospatial datasets of human settlements, the physical environment comprising geology, land cover, hydrology, and infrastructure such as transportation and utility networks. To assess the feasibility of each solution, capital and operational costs as well as operational sustainability calculations were established within the model. The subsequent coding has been carried out to initially test the programme on four counties (Wexford, Leitrim, Sligo and Limerick) within Ireland. The ArcGIS tools and functions that have been applied for each strategy evaluation will be explained in the results section. The model is set as a web service on the Amazon EC2 cloud as a simple, scalable and independent rich internet application. It was developed using ESRI ArcGIS Server 10.1 and ArcMAP 10.1 and will be configured as a thin client/server application. The model will be placed as a web service on the ArcGIS server and exposed to the thin client through the Representational State Transfer (REST) protocol.

RESULTS AND DISCUSSION

The results show that potentially polluting sites represent 32% up to 84% of all existing OSWWTSs in the different counties thus highlighting the need for such a decision support tool as part of an appropriate managing system. A modelling architecture (Fig. 1) has been developed that takes as its initial input the location of a proposed (or existing) house within an area of low permeability subsoils. The proposed six solutions and their suitability for the selected site are then evaluated in parallel. While the on-site solutions are always included as suggested options, the selection of other appropriate solutions depends principally on four major model parameters: distance from an existing sewerage network; existing OSWWTS density (for reasons of economies of scale); the distance to surface water; and the depth to bedrock. Through a series of interconnected GIS geoprocesses the model outputs appropriate solutions for a site, ranked in terms of sustainability and cost. However, it should be noted that any final decisions are still dependent on on-site constraints. Therefore, each solution is accompanied by an alert message that provides additional information to refine the output list according to the available local site specific information.

Connection to existing sewer network

The ArcGIS Network Analyst was used to determine the road distance from a selected site to the edge of the closest sewered area. This tool was selected over a standard buffering or distance function in order to estimate the length of the required sewer connection along the road, with associated installation costs. The maximum distance of 100 m (less than which the connection to an existing sewer network is considered a viable option) was chosen initially but can be changed according to the Local Authority's needs. Results from GIS analysis for the four test counties show that between 5% and 9% of all potential legacy sites lie within a 100 m radius distance of an existing sewer network. These proportions increase accordingly when a larger radius distance around sewered areas is considered providing a potential solutions to 3400 (100 m radius) up to 6000 houses (250 m radius) within the four counties. However, the additional expenses for extending the sewerage network will need to be considered. It should also be noted that while the programme might suggest the connection to an existing sewer network as a viable solution, the available treatment capacity for the specific treatment facility will still need to be assessed, as indicated by an appropriate alert message.

Clustering of houses with decentralised wastewater treatment and surface water discharge

The ArcGIS kernel density function was calculated in order to determine whether it would be feasible to connect several houses via a small bore sewer network that feeds a decentralised treatment plant with a licensed discharge to surface water. If the OSWWTS density in the area of a selected site is greater than 16 systems/km^2, an iterative buffering and clipping sequence is used to identify houses that are close enough to be connected up with a small decentralised sewer system. In order to keep the bore sewer length (and therefore costs) to a minimum, only houses that are within 80 m distance of each other are included in the cluster. A buffer of 150 m around the final cluster is then intersected with the river polygon layer to find a potential surface water discharge point. Where this is not given, the option of a decentralised wastewater treatment system is dismissed. Based on this approach clusters comprising at least 4 houses were identified in areas of high likelihood of inadequate percolation within the four test counties (Table 1). Generally more clusters were found in denser populated counties such as Limerick and Wexford, respectively. Equally, the average cluster size varies between counties. Only half of the identified clusters (44.3% - 58.4%) lie close to a river and could be considered for this solution. These clusters represent 12.8%, 13.1%, 33.2% and 33.9% of all legacy sites in areas of low subsoil permeability in Sligo, Leitrim, Limerick and Wexford respectively. Over 80% of clusters comprise of less than 21 houses but cluster sizes will increase when a larger maximum distance between houses (e.g. 100 or 120 m) is considered. The economic feasibility of increasing this parameter value within the programme still remains to be tested.

Overall these results show that decentralised treatment could provide a solution particularly for more densely populated counties such as Limerick and Wexford. However, before the proposed discharge from such a decentralised wastewater treatment plant to surface water can be considered, the assimilative capacity of the proposed receiving river needs to be assessed (which the user is again notified by an on-site constraint alert).

Discharge onto bedrock through imported media filter

Another option is to evaluate whether the depth of low permeability subsoil is shallow enough to be replaced by a more suitable imported media (soil or sand) through which treated effluent will percolate down into the bedrock. A maximum depth of 3 m was considered to be economically realistic for the excavation of the existing subsoil. Hence, the depth to bedrock for the selected site location was obtained from GIS maps. Figure 2 shows the number of houses located in areas with high likelihood of inadequate percolation for which this solution would be potentially suitable. These represent 5.1% and 4.9% of the legacy sites in Counties

Leitrim and Wexford but only 2.5% and 1.4% in Sligo and Limerick, respectively. Knowledge about the type of bedrock present underneath the shallow subsoil can give an indication of the expected hydraulic properties. Hence, Figure 2 includes information on the bedrock type (obtained from available GIS maps) that can be expected at the considered sites. It shows that about half of the sites in Leitrim and Sligo are underlain with potentially impermeable metamorphic rocks such as quartzites, gneisses and schist but both counties also have sites on permeable bedrock such as sandstone and limestones. However, the actual permeability is largely affected by the depth and extent of fracturing and weathering so that an individual site assessment to determine the local bedrock permeability as well as the water table depth might be inevitable.

On-site treatment systems

The remaining on-site solutions that would be suitable at a single house scale are always considered by the programme as the suitability of these systems is mainly dependent on site specific constrains (e.g. subsoil permeability and available space at the property) that are not available from GIS maps. Hence the user will be given additional information together with the suggested solution that enables the user to refine the output list according to local site specific information.

Cost and sustainability ranking

To provide the opportunity to evaluate the alternative disposal options on the basis of both cost-benefit and environmental sustainability principles, calculations to estimate capital and operational costs as well as greenhouse gas (GHG) emissions associated with each solution have been established. Capital cost estimations comprise material and installation costs, including the labour required for the systems installation. Operational costs include operation costs (resulting mainly from the systems electricity usage) as well as maintenance costs that arise for the householder for system services and desludging. Environmental sustainability is only estimated for the systems' operation and is primarily based on the CO_2 emission related to the electricity usage and the diesel usage for desludging. All costs are calculated with and without the use of water saving devices which have been shown to reduce a household's wastewater production and hence both capital and operational costs of certain disposal systems (Dubber and Gill, 2013). Cost savings related to the water and energy savings achieved by water saving devices are charged against operational costs. A similar approach is used for the assessment of operational GHG emissions.

All cost and emission calculations integrate and use results from GIS based computational tasks as input parameters for their estimations. Examples are the road distance of a house to the nearest sewer network, the size of suggested houses cluster, the length of proposed decentralised sewer networks and the distance from a house to the next WWTP for sludge or wastewater disposal. Hence, the costs and environmental sustainability rankings for the different solutions are site specific. However, general trends show that evapotranspiration systems are one of the most expensive solutions in terms of capital costs, even with the use of water saving devices, but operational costs and GHG emissions are the lowest, especially when designed as a gravity flow system. In addition, the energy savings from water saving devices and associated lower water heating will reduce the household's CO_2 footprint.

CONCLUSIONS

The developed decision support toolset can be used by Local Authority planners and managers to assess the feasibility of different sewage treatment and disposal systems for new developments and existing houses in rural areas with low permeability subsoils. It should be noted that the model is designed to be a decision support tool and that any final decisions taken would obviously still be dependent on on-site constraints to refine the output solutions. However, this will help to improve the management of on-site wastewater treatment and consequently help to protect ground- and surface water from faecal pollution and eutrophication, protecting water resources and improving human health.

ACKNOWLEDGEMENT

Acknowledgment is given to the Irish Environmental Protection Agency for funding this research.

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Table 1. Clusters identified in the four test counties. The maximum distance between houses in the cluster is 80 m and only clusters containing a minimum of 4 houses are considered. Numbers in brackets represent clusters with a river within 150 m distance.

	Leitrim	Limerick	Sligo	Wexford
Number of clusters	197 (100)	469 (274)	230 (102)	928 (411)
Max. cluster size	196	592	164	289
Average cluster size	13 (15)	23 (30)	11 (13)	14 (18)
Number of houses in all clusters	2,469 (1,474)	10,580 (8,251)	2,517 (1,298)	12,860 (7,229)
Number of legacy sites in clusters	2,178 (1,226)	4,860 (3,392)	2,182 (1,094)	10,843 (6,058)
% of legacy sites within county	13.1	33.2	12.8	33.9

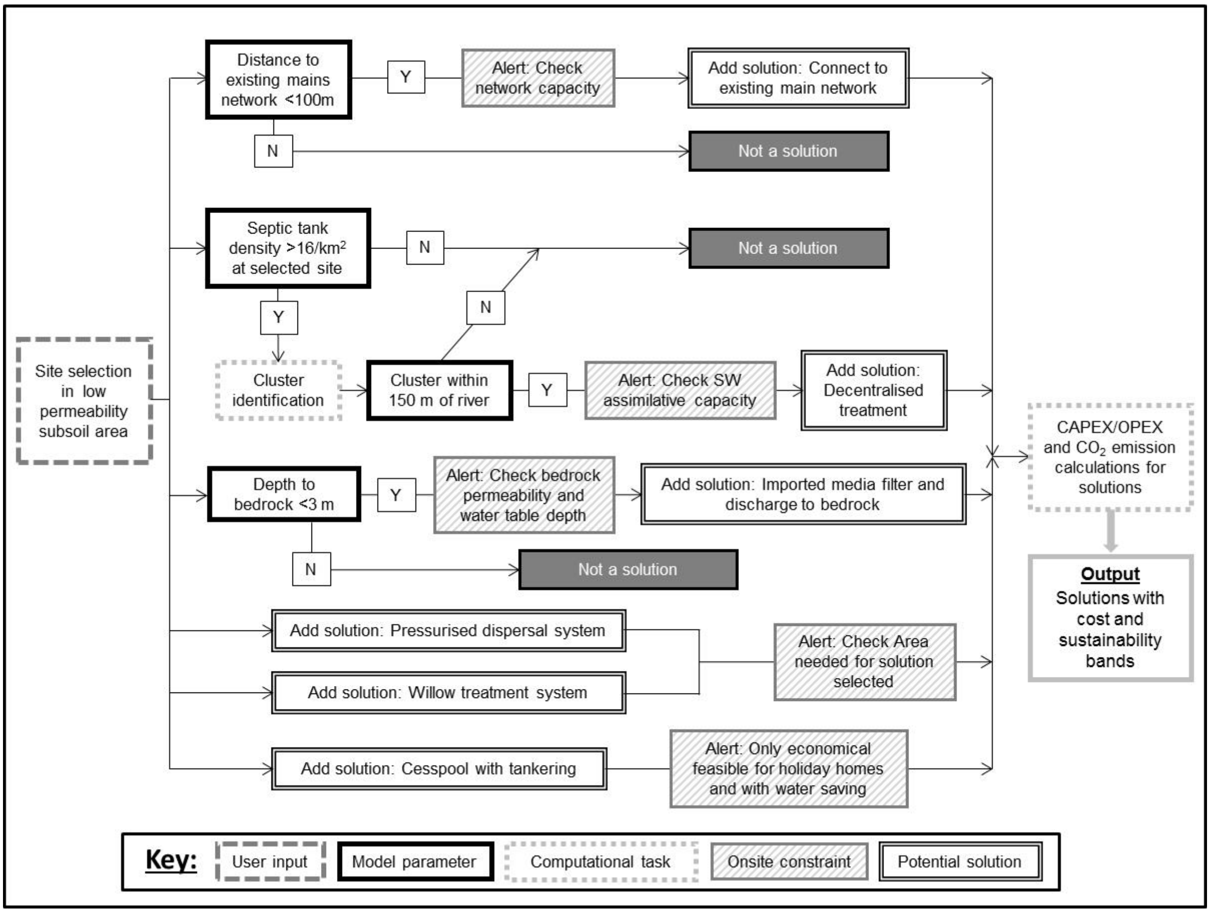

Figure 1 Modelling architecture for the GIS based decision support toolset

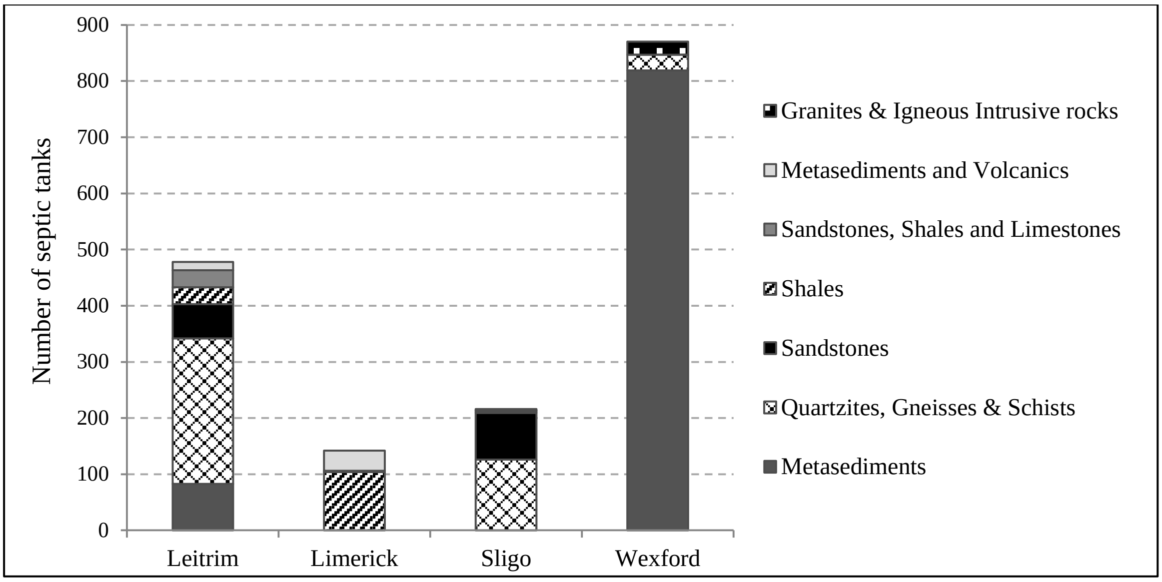

Figure 2 Number of septic tanks for which discharge to bedrock would be considered within the four test counties and the expected bedrock types at those sites.

Water quality tool set for coastal Georgia onsite wastewater treatment system planning

Ray Bodrey

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ABSTRACT

The population in coastal Georgia is growing at a significant rate. Coastal cities have limited funding and time to upgrade their municipal treatment plant infrastructure to counter the rise in population. Therefore, onsite wastewater treatment systems (OWTS) have and will continue to be heavily permitted. These systems have the potential to impair water quality if not maintained, leaching bacteria and nitrogen that may cause health risks to both humans and the environment.

With growing concerns, it is extremely important to establish a water quality assessment plan to enable better public health planning. The University of Georgia Marine Extension Service and project partners conducted a survey of geo-locating and inspecting on-site disposal systems in proximity of state waterways of coastal Georgia. The data was transferred to the WelSTROM (Well and Septic Tank Referencing and Online Geo-location) GIS database. From information analyzed through this database, a pilot study has begun to assess water quality in areas of varying OWTS densities within Glynn County, GA. A nonpoint source transport model will be developed from utilization of project data. Another study has begun to determine nitrogen fate and transport in coastal Georgia soils on a mounded onsite wastewater treatment system, a new technology implemented in the region. The soils project will produce a 2-D Hydrus model. The process of geo-locating systems, evaluating water quality in system densities and the development of both a water quality and soil transport model will create a powerful toolset in determining pollution susceptibility in Coastal Georgia.

INTRODUCTION

OWTS Geo-location & Analysis Project

According to the NOAA's State of the Coast website, over 120 million people (39% of the U.S. population) live along the U.S. coast on only 18% of the nation's land mass. Coastal populations are projected to steadily increase and coastal Georgia is not immune to this trend; this area is one of the fastest growing in the state.

A major challenge for environmental professionals is the issue of nonpoint source water pollution. Continuing urban sprawl has made water quality issues a primary concern. The central issue regarding population growth in this manner is stormwater runoff coupled with OWTS failure. Most of Georgia's coastal counties have limited public sewage treatment infrastructure and rely heavily on OWTS to handle human sewage production.

The US EPA/NOAA Findings and Conditions Report noted that coastal Georgia was deficient in several areas of nonpoint source pollution controls, especially the management of onsite septic disposal systems (US EPA/NOAA, 1999). "Of Georgia's 20 estuarine areas, 19 are closed to shellfish harvesting. Ten of these areas are closed due to fecal contamination from nonpoint sources" (Georgia Department of Natural Resources, 1998; p. 67). In an effort to mitigate these conditions, in 2002, The University of Georgia Marine Extension Service (MAREX) developed a partnership with the Georgia Environmental Protection Division (EPD), Coastal Health District of Georgia (CHD) and the Southern Georgia Regional Commission (SGRC). MAREX served as the main project coordinator for this partnership to implement a Clean Water Act Section 319 (h) grant supported project to geo-locate, inspect, inventory and analyze OWTS in areas of critical concern along Georgia's coast. The initiative lasted 4.5 years and spanned eleven counties. The deliverables obtained will assist the CHD and other governmental agencies and private organizations with better public health planning as well as providing valuable data concerning emergency management plans, disaster-resiliency and coastal hazards risk assessments.

OWTS Density Water Quality Evaluation

Building upon the success of the 319 (h) initiative, MAREX was awarded a GA Department of Natural Resources Coastal Inventive Grant to fund a pilot study in Glynn County, GA to further research the link between OWTS pollution and Georgia's coastal waterways. The primary goal of this ongoing project is to conduct water quality analysis in areas of possible pollution attributed to nonpoint source, specifically areas of OWTS densities geo-located in the 319 (h) project. Specifically, our goal would help address this issue raised by CSREES (2004), "Concerns have been raised that combined output from densely packed onsite wastewater treatment systems may exceed the natural ability of soils to receive and purify the wastewater before it reaches groundwater or adjacent surface water".

By gathering and analyzing water quality data, this project will provide significant insight into pollution prevention and protection of water bodies facing potential impairment in the region. The two primary objectives of this project are to sample and analyze water quality in areas of selected OWTS densities and develop a transport model for government officials, and public and private environmental professionals to utilize for planning purposes.

Nitrogen Fate & Transport in a Coastal GA Mounded OWTS

Finally, the third project in this study, is a soil water chemistry project investigating the nitrogen fate and transport in a mounded OWTS in Glynn County, GA. Soils in coastal Georgia tend to be unsuitable for septic drainfields, due to poor percolation rates. Therefore, mounded systems are often permitted to combat this problem. Though mounded systems have been used in other states for decades, these systems are a relatively new wastewater treatment technology in coastal Georgia.

Limited research has been done on mounded systems in a coastal Georgia application. With nitrogen being a primary function in nonpoint source pollution activity for marine waters, a study was needed to determine nitrogen transport and fate in coastal soil environments. Recently, a study was designed and implemented in northern Georgia, where a nitrogen model was develop utilizing a conventional OWTS and piedmont soils (Radcliffe, Bradshaw, 2013). The project processes have been adopted for our coastal Georgia mounded system research study.

MATERIALS AND METHODS

OWTS Geo-location & Analysis Project

MAREX supplied Georgia coastal county environmental health inspectors with Trimble Juno SB handheld GPS units and provided training on the operation of the GPS unit and data transfer process as well as a proposed strategy for the field work. In each county, inspectors geo-located and visually inspected relevant parcels for OWTS tanks/drainfields and wells in potentially critical areas near state water bodies. Each county was provided with a strategic map for data collection purposes highlighting critical areas. Each inspector visually examined all septic tank systems for signs of failure and, if found, noted the system as failing or suspected of failure in the GPS unit's data log referencing that particular point. The points gathered were periodically uploaded into the WelSTROM GIS database, which houses well and septic tank reference data along with a GIS component for geo-location.

Twelve maps were also created for each county involved. Each map consisted of a particular GIS layer with geo-located septic system points imposed. Some examples of the map layers are Floodplain data (FEMA), Impaired Waters (Georgia Environmental Protection Division), National Wetlands Inventory Data (U.S. Fish & Wildlife Service) and Ground Water Recharge Zones (Georgia Department of Natural Resource). Please see figure section for an example of Chatham County's STATSGO Soil map (NRCS) (Figure 1).

Pollution Susceptibility Index

The pollution susceptibility index methodology was created through a workgroup process with oversight from scientists, planners, environmental health professionals and government officials. The SGRC, utilizing the methodology, further developed the index by utilizing ArcGIS. ESRI ArcGIS 10 with Spatial Analyst extension was used for all GIS processes. The septic pollution susceptibility index was created by a varying weighted value system of the GIS risk factor layers. Please see reference section for risk factors/weighted values (Table 1).

OWTS Density Water Quality Evaluation

MAREX developed a surface water sampling plan based on the densities of septic system locations found in the previous Phase II OWTS grant project for Glynn County. MAREX is sampling 10 stations/monthly, surface water grabs, for 24 months and analyze the water quality in reference to nonpoint source pollutants and indicators. MAREX worked closely with Glynn County Environmental Health Department of the Coastal Health District to select sampling stations, some of which are in proximity to historical septic system failures/repairs and some are in areas of limited exposure to land development. Analysis parameters consist of a standard profile including dissolved oxygen, current, pH, salinity, temperature, turbidity and visibility. Nutrient parameters analyzed include ammonia-N, phosphates, nitrate-N and nitrite-N; using a Lachat Analyzer. Bacteria parameters analyzed include fecal coliform bacteria (A-1 method) and enterococci (membrane filtration). If certain stations are shown to be "hot spots" in year one of the study, a bacterial source tracking method will be used to identify either human or wildlife source. Chlorophyll and biological oxygen demand are also factors in the analysis.

The laboratory data, along with precipitation and tidal data will be used to create a transport model. The statistical data will be projected in GIS, over the project period in order to focus on trends. It's not only important to know the status of nonpoint source pollution in the selected waterways, but equally important to know the movement of the contaminants via the transport model. Pollutants may be either dissipating, being pulled into back creek areas, pushed out onto the deeper water Continental shelf, or a combination of these transport theories, depending on a number of variables (current, turbidity, rainfall, storm surge, tidal range and other data). The model will seek to answer these questions regarding transport.

This plan will assess whether water quality parameters measured are below TMDL standards for the region. If this is shown to be below standards, a recommendation will be made in the final report of the project. This recommendation will likely be that of a source tracking plan. The plan most likely would be formulated utilizing a tracer material and implemented in a state or county owned OWTS in the area to determine if increases in parameters measured are in fact originating from OWTS and not stormwater activity.

Nitrogen Fate & Transport in a Coastal GA Mounded OWTS

This project will encompass a nitrogen modeling application by using the HYDRUS software. The key to controlling eutrophication in freshwater systems is managing phosphorous inputs. Conversely, the key to controlling eutrophication in marine systems is managing nitrogen inputs. Dr. David Radcliffe from UGA Crop & Soil Science, along with Ken Bradshaw, former PhD student, have developed a nitrogen model regarding conventional septic systems in northern Georgia regarding piedmont soil types (Radcliffe and Bradshaw, 2013). This project proposes to use this modelling application on a mounded system in coastal Georgia soil types and model nitrification and denitrification activities.

With the help of the Glynn County Environmental Health Department and the Coastal Health District, a research site has been selected at the county maintained, Blythe Island Regional Park Campground. This mounded system is 5,680 liters per day maximum (1500 gpd) and is heavily used, with an estimate of 2,650 liters a day (700 gpd) average usage. The system is located in the area of the water quality sampling stations defined by the Coastal Incentive Grant.

Installed on the drainfield are the lysimeters, tensiometers, an automatic rain gauge, a flowmeter and control boxes (with Campbell Scientific CR-1000 data loggers). Ten nested pairs of tensiometers will be placed at 60 cm (24 inch) and 107 cm (42 inch) depths and will measure soil water tension. Essentially, 60 cm depth is in the lower drainfield as the 107 cm depth is in native soil. Lysimeters will be placed at 107 cm depths (42 inches) with each nested pair of tensiometers. The lysimeter will be the collection point for the nitrogen sample. The equipment used to monitor the site has been graciously provided by Dr. Radcliffe and Dr. Mark Risse, Director of UGA Marine Outreach Programs.

For a period of one year, the site will be visited every 2 weeks to check on the equipment and download data from the CR-1000s. A water sample from each lysimeter will be collected twice a month and analyzed for nitrate/nitrite utilizing the Lachat Analyzer . This data will be analyzed with Hydrus Software, in order to produce the model depicting nitrogen transport.

RESULTS AND DISCUSSION

The OWTS Geo-location & Analysis Project is now complete, regarding the grant processes and funding timeline. The OWTS Density Water Quality Evaluation project is now in month 6 of 24. The Nitrogen Fate & Transport in a Coastal GA Mounded OWTS project is being implemented at this point, however the monitoring system is not yet operational. Results are expected to be published in 2015.

The OWTS Geo-location & Analysis, including the phase III project managed by the SGRC, has mapped approximately 25 thousand OWTS parcels. The initiative produced the first GPS inventory of septic systems and wells in the 11 Coastal NPS region and improved collection and verification of OWTS position locations followed by data entry into the Department of Community Health's mandatory Statewide Digital Health Department Database. The development of geo-location capacity (WelSTROM) has improved local and state management of septic systems and wells for coastal Georgia. The project has also produced GIS maps and analysis of the OWTS and wells utilizing the SGRC's geo-referenced WelSTROM GIS database which are all web accessible for better public health planning.

The WelSTROM database provides a standardized method of recording all current and future OWTS installations for the eight counties of the Coastal Health District. A pollution susceptibility index for each of the coastal health district counties was also created. See reference section for an example of the Glynn County Pollution Susceptibility Index (Figure 2).

CONCLUSIONS

Once complete, this initiative will have provided a GIS database of OWTS, an evaluation of surface water quality in selected densities of OWTS, and an evaluation of nitrogen fate and transport in coastal soils. This tool set can be utilized by government officials, planners and scientists to better articulate and quantify water quality conditions in the watersheds. The products can be used to assist in the identification of potential pollution sources and as an analytical tool to aid in establishing TMDL processes required for coastal waters.

Moreover, data from the project should be shared with state and local governments regarding hazard resiliency, emergency management planning and other vulnerability assessments. It is also imperative that geo-located OWTS data is used in reference to flood and natural disaster emergency response plans.

Project reports, maps and pollution susceptibility indices for coastal Georgia can be downloaded at: http://marex.uga.edu/water_quality/

WelSTROM GIS database site: http://www.sgwebmaps.com/welstrom/

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TABLES AND FIGURES

Risk Factor	Category	Value
FEMA floodplains	In 100 yr. floodplain area In 500 yr. floodplain area Not in floodplain	50 30 0
Proximity to Wetlands	0 to 500 ft. from wetlands	100 to 0
Pollution Susceptibility (DRASTIC)*	Medium High	50 100
Groundwater Recharge Areas	Within recharge area Not within recharge area	50 0
Proximity to Shellfish Beds	0 to 500 ft.	100 to 0
Proximity to 305(b)/303(d) Impaired Streams/Lakes/Sounds	0 to 1000 ft. from Impaired Stream centerline; 0 to 500 ft. from Impaired Lakes/Sounds	100 to 0
Proximity to Surface Waterbodies	0 to 500 ft. from surface waterbody	100 to 0
OSDS density	low density to high density	0 to 100
TMDL Impaired Watershed	Within impaired watershed Not within impaired watershed	50 0

Table 1. Risk Factors and Weighted Values for Pollution Susceptibility Index.

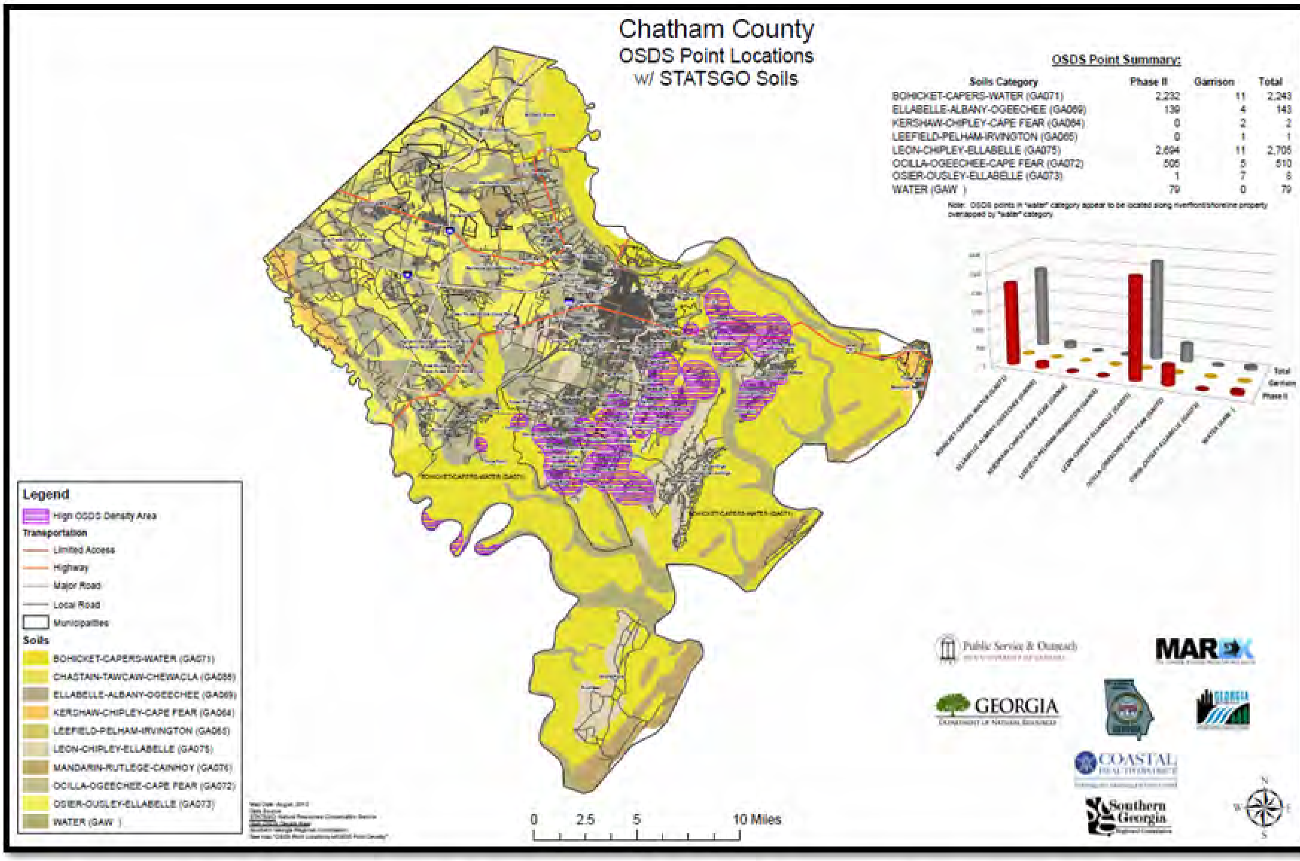

Figure 1. Chatham County, GA OWTS Locations and STATSGO Soil (NRCS) GIS layer.

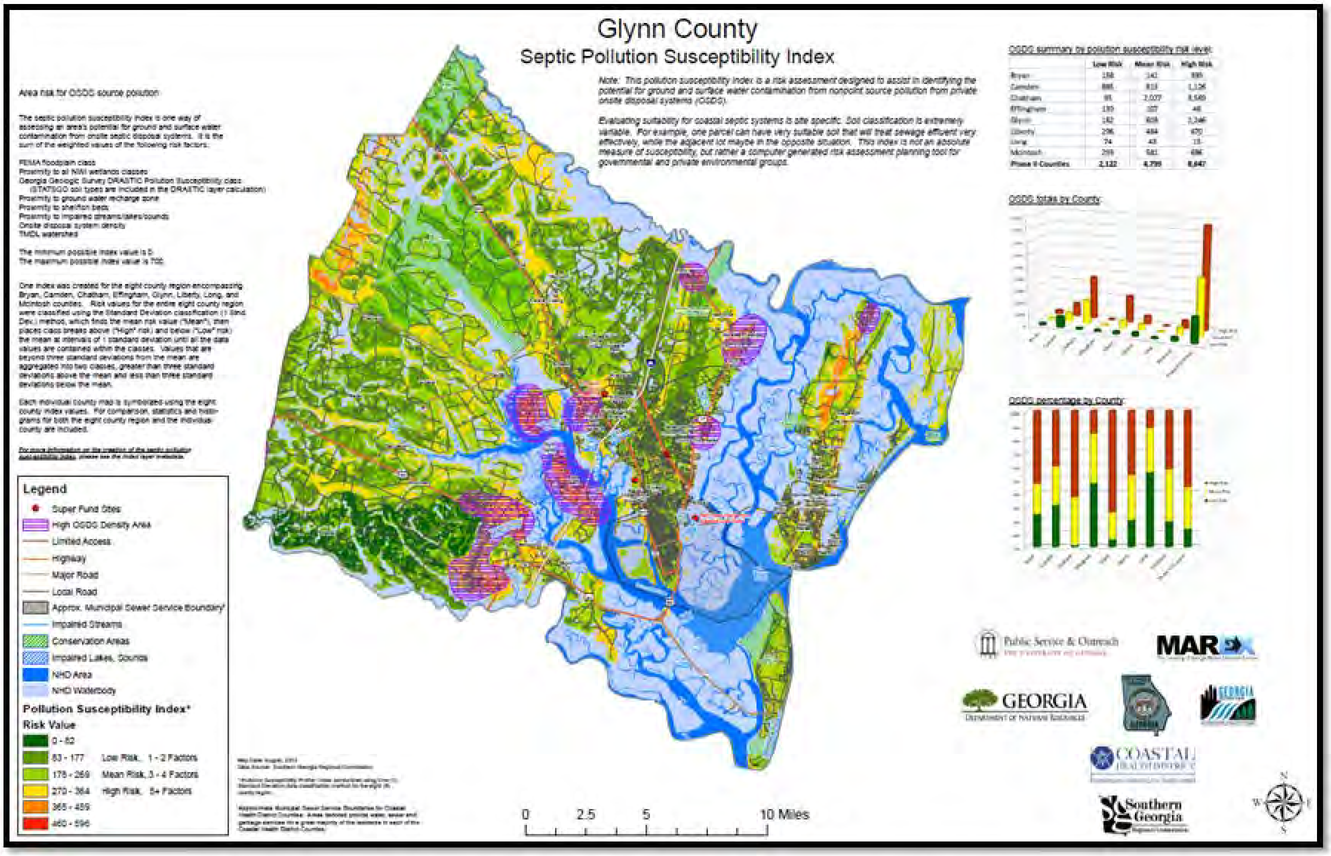

Figure 2. Glynn County, GA Pollution Susceptibility Index.

Capacitively-Coupled Resistivity Surveys to Delineate Subsurface Wastewater Migration in Coastal Surficial Aquifers.

Michael O'Driscoll, East Carolina University

ABSTRACT

Nutrient exports from conventional onsite wastewater systems to surface waters are not well-constrained. There is a growing need to better quantify non-point source nutrient inputs to surficial aquifers and surface waters, particularly in nutrient-sensitive coastal watersheds. Characterizing subsurface wastewater migration and associated nutrient transport in coastal surficial aquifers can be challenging and resource-intensive due to the need for site access, invasive soil and hydrogeological site characterization, in-situ groundwater monitoring, and water quality analyses. Since wastewater typically has elevated specific conductivity, shallow aquifers that receive wastewater may respond by becoming more conductive (or less resistive) to the flow of electrical current. Capacitively-Coupled Resistivity (CCR) surveying is a technique that quantifies subsurface electrical resistivity. This technique can be useful in onsite wastewater studies because it can potentially detect changes in groundwater conductivity that can be used to quantify the extent of subsurface wastewater migration in shallow aquifers. During the course of three groundwater investigations, we evaluated if CCR surveys could help quantify the extent of wastewater transport in the subsurface. The onsite wastewater study sites included schools, an environmental education center, and private residences in Pitt, Beaufort, and Craven Counties, located in the Coastal Plain of North Carolina. Electrical resistivity surveys were conducted with an OhmMapper (Geometrics, Inc.) and apparent resistivity data was inversely modeled using RES2DINV and RES3DINV software. Soil and sediment cores were collected adjacent to resistivity transects. Groundwater environmental measurements taken at the survey sites included depth to groundwater, pH, specific conductivity, temperature, dissolved nitrogen, and dissolved oxygen. Water samples were collected from septic tanks, groundwater beneath the drainfield, groundwater up and down-gradient from the onsite wastewater system, and from nearby surface waters. We compared electrical resistivity, groundwater specific conductivity, and dissolved nitrogen concentrations in the surficial aquifer adjacent to onsite wastewater systems. Overall, the results showed that CCR surveys were sensitive to the presence of wastewater in sandy surficial aquifers with shallow water tables (< 5 m deep) and were able to detect changes in groundwater specific conductivity at depths of up to approximately 8m. Resistivity data can help delineate the orientation and extent of subsurface wastewater plumes if the contrast between background groundwater and wastewater-affected groundwater is greater than approximately 200 mS/cm. Interpretation is most straightforward when the sandy surficial aquifer sediments are relatively homogeneous because clay layers can also result in lower resistivity values.

Spatial Distribution of Wastewater Microbial Indicators in Groundwater Beneath Two Large Onsite Wastewater Systems.

Charles Humphrey, East Carolina University

ABSTRACT

Unequal distribution of onsite wastewater system (OWS) effluent to drainfield trenches may lead to hydraulic failure and/or decrease wastewater treatment efficiency. Therefore equal distribution is important for OWS performance. The study objective was to assess the spatial distribution of microbial indicators (coliform, enterococcus, and E. coli) in groundwater beneath two large OWS. Both OWS used dual-alternating drainfields and were in operation for more than 12 years prior to the study. One system used a pump to distribution box system (D-box), while another used a low pressure pipe (LPP) distribution. Monitoring wells were installed up and down-gradient from the OWS drainfields. Monitoring wells (4) were also installed near the front of the trenches and 4 wells were installed near the end of the trenches for each system. The wells were evenly spaced across the two drainfields at each site. Water samples were collected 4 times from each of the wells, septic tanks, and nearby stream for coliform, E. coli and enterococcus analysis using the IDEXX method. The pH, electrical conductivity, and temperature of the samples were determined using field meters. Groundwater physical, chemical, and biological parameters at the front of the trenches were compared to characteristics near the end of the trenches for both systems to help determine if effluent was more uniformly distributed via LPP or D-box. Results indicate that both systems were effective at reducing indicator bacteria concentrations (all > 99%) before discharge to groundwater. Enterococcus concentrations were significantly higher in groundwater near the end of the trenches (in comparison to the front) for the OWS with D-box distribution, but not for the OWS with LPP distribution. Moderate to weak correlations between enterococcus and total coliform were observed at both sites. Overall, groundwater biogeochemistry was more similar beneath the LPP system in comparison to the pump to D-box.

Evaluation Of Water Quality Renovation By Advanced Soil-Based Wastewater Treatment Systems.

Jennifer Cooper, University of Rhode Island

ABSTRACT

25% of US households utilize onsite wastewater treatment systems (OWTS) for wastewater management. Advanced technologies were designed to overcome the inadequate wastewater treatment by conventional OWTS in critical shallow water table areas, such as coastal zones, in order to protect ground water quality. In addition to the septic tank and soil drainfield that comprise a conventional OWTS, advanced systems claim improved water renovation with the addition of sand filtration, timed dosing controls, and shallow placement of the infiltrative zone. We determined water quality renovation functions under current water table and temperature conditions, in anticipation of an experiment to measure OWTS response to a climate change scenario of 30-cm increase in water table elevation and 4°C temperature increase. Replicate (n=3) intact soil mesocosms were used to evaluate the effectiveness of drainfields with a conventional wastewater delivery (pipe-and-stone) compared to two types of pressurized, shallow narrow drainfields. Results under steady state conditions indicate complete removal of fecal coliform bacteria, phosphorus and BOD by all soil-based systems. By contrast, removal of total nitrogen inputs was 16% in conventional and 11% for both advanced drainfields. Effluent waters maintained a steady state pH between 3.2 – 3.7 for all technologies. Average DO readings were 2.9mg/L for conventional drainfield effluent and 4.6mg/L for advanced, showing the expected oxygen uptake with shallow placement of the infiltrative zone. The conventional OWTS is outperforming the advanced with respect to nitrogen removal, but renovating wastewater equivalently for all other contaminants of concern. The results of this study are expected to facilitate development of future OWTS regulation and planning guidelines, particularly in coastal zones and in the face of a changing climate.

The Past 100 Years and Future of On-site Resource Water

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ABSTRACT

The history of effluent treatment on-site prior to the 1950s provides an interesting window into what could work and what might be sustainable for individual homes and small communities into the future. In 1894, George E. Waring, Jr. stated, "It has hitherto been – and, in fact, still is – the practice of the world to consider its wastes satisfactorily disposed of when they are hidden from sight. In spite of an almost universal outcry about sewer-gas, filth diseases and infective germs, the great mass, even those who join in the cry, pay little heed to defects in the conditions under which they are living so long as they are not reminded by their eyes or their noses that their offscourings are still lurking near them." Early references show much thought and consideration about flow control, filtration, aerobic treatment, shallow soil dispersal, various loading rates, high strength waste treatment and maintenance. The relationship between disease-causing organisms and proper handling of sewage were understood and will be explored. Furthermore, infrastructure independence and sustainable practices, such as water and nutrient recycling, will be discussed.

INTRODUCTION

The need for on-site treatment systems that are sustainable and protective of the public health and the environment has long been recognized. The United States Department of Agriculture (1896) indicated that the goal for farmers was to protect clean water sources. "The vital thing which thus presents itself is the disposal of fecal matter and other refuse so that the wells, upon which most rural families depend for their drinking water, may remain pure." On-site treatment system sustainability has been an evolutionary process, typically constrained by the technological limitations of the era. The other factors in the sustainability equation are the end-user (homeowner) and the service provider, which historically was the end-user. The United States Department of Agriculture (1928) recognized this relationship. "Care in operating is absolutely necessary. No installation will run itself. Continued neglect ends in failure of even the best-designed, best-built plants. If the householder is to build and neglect, he might as well save expense and continue the earlier practice." In this case, the "earlier practice" was a cesspool, rather than a septic tank and drainfield (for the time, a technological "step-up").

Hardenbergh (1924) also recognized this relationship. "The average city plant is operated by a skilled attendant; the average home plant receives practically no attention. Reserve capacity should be provided to care for these factors, and sufficient storage provided to equalize the abnormal hourly flows and allow a certain minimum retention period for the sewage. The installation should be as nearly automatic as possible, and should be designed to operate practically without attention."

MATERIALS AND METHODS

In 1877, Schlössing and Muntz "demonstrated that oxidation in soils is due to an organized ferment." An organized ferment could be described as the treatment processes that occur through microbial biofilms. They found that “sewage, slowly filtered through a column of sand of sufficient depth, was completely purified. If chloroform was introduced, essentially benumbing the organisms in the sand, no purification took place until the effect of the chloroform had passed away. They accepted this as proof—and later knowledge confirms it—that purification is due to living organisms” (Waring, Jr., 1894).

The impact this discovery would have on public health can perhaps be best understood by examining the consequences of its absence. Parry, 1929 comments, “Probably no epidemic in this continent’s history better illustrates the dire results that may follow one thoughtless act than the outbreak of typhoid fever in Plymouth, PA, in 1885. In January and February of that year, night discharges (urine or feces likely collected in a bedpan or chamber pot) of one typhoid fever patient were thrown out in the snow near his home.” Pathogens in those discharges, carried by spring thaws into the water supply, caused an epidemic that lasted from April to September. In a total population of 8,000 people, 1,104 citizens contracted typhoid fever and 114 died (Parry, 1929).

Researchers in Birmingham, AL tracked annual typhoid fever deaths from 1910 to 1922. Prior to sanitary surveys being conducted and the installation of 6,000 sanitary privies in homes not yet reached by sewers, there was an average of 48 deaths per 100,000 people. A sanitary or pit privy is a shallow dug hole with a toilet seat structure inside a small enclosure. They are often referred to as outhouses. Following the installation of the sanitary privies, the mortality rate dropped by 52 percent to an average of 23 per 100,000 people. There were similar findings in a study conducted in Berkeley County, WV, from 1911 to 1917 and Yakima, WA, from 1908 to 1914 (Hardenbergh, 1924).

In England, Robert Warington did work on nitrification. He “proved conclusively that the oxidation of ammonia and organic matter was effected by the agency of living organisms, and Warington proceeded to devise practical methods whereby living organisms could be utilized for the nitrification of the organic matters in sewage. (Metcalf and Eddy, 1930)”

Once the role untreated sewage played in public health was understood, communities across the continent began increasing the use of septic tank systems at the individual household level. Guidelines governing the design and installation of systems were established along with target performance standards. Hardenbergh (1924) notes, “To operate properly and to prevent pollution of the ground or the ground water, septic tanks should be water tight. Any material is permissible, so long as it is durable and does not leak.”

In 1926, the Sewerage and Irrigation book addressed uniform or timed dosing: “If the tank is supplied at a uniform rate with material of the same composition, a uniform condition will be established. If the composition of the supply or the rate at which it passes through the tank is varied, the bacterial life, and consequently the character of the effluent, will vary. (International Library of Technology 440, 1926)”

In 1927, Septic Tanks for the Farm noted referenced soil-based treatment: “By far the greater number of bacteria is in the upper foot or two of soil. (Haswell, 1927)"

RESULTS AND DISCUSSION

Little difference exists in how an on-site treatment system, typically a septic tank and gravel-filled drain field, looks or functions today versus 100 years ago. The septic tank was first introduced in the late 1800s. Folwell (1910) noted that "the true function of a septic tank is to remove and hydrolyze the suspended matter" with the thought that "the sewage should not stay too long in a septic tank, from six to 12 hours being found best." Also noteworthy is "the effluent of a septic tank is therefore in better condition for disposal by dilution than merely settled effluent. Moreover, the grosser matters, which cause surface clogging of filters, are removed. It is a question, however, whether the septic effluent is better adapted for disposal on fine-grain filters, as the fineness of the suspended matter and absence of the surface mat, which is formed on a filter when coarser matters are present, result in a deeper penetration of the deposits" (Folwell, 1910).

There were different methods for drain-field construction, depending on whether the land was flat, gently sloping or steeply sloped. Tightly packed soils were supposed to be deeply subsoiled and underdrained with the intent of constructing a deeper trench. Porous, well-drained, air-filled soil is an absolute necessity. Subsoiled ground should have three- to four-inch distribution tile, with the depth varying from 1.25 to 3.5 feet (0.38 to 1.06 meters). If planting crops over the drain field, the depth should be 3.5 to 4 feet (1.06 to 1.21 meters) deep (USDA, 1928).

The Portland Cement Association (1937) was promoting concrete septic tanks as a way to guard the health of families. It advertised the septic tank as the "bulwark of safety". An early advertisement shown in Fig. 1 and Fig. 2 touted the benefits of the septic tank (Ogden and Cleveland, 1913).

As early as the 1920s, septic system designers for commercial applications were aware of the need to treat grease differently than other wastes and were building traps and tanks for oil and grease removal. As such, today we know how to better deal with organic (biochemical oxygen demand [BOD]) loading, hydraulic loading, fats, oils and grease (FOG) loading and chemicals (e.g., quaternary ammonium compounds [QAC]) in on-site treatment systems. For example, a low QAC concentration is highly detrimental to biological treatment processes. Therefore, service providers need to know the QAC concentration in a food service establishment waste stream. QAC concentration can be measured using test strips or estimated using a spreadsheet.

Kinnicutt et al. (1910) describes the practice of community irrigation and "sewage farming" in England and the United States. Today, we refer to this practice as water and nutrient capture and reuse. Areas with successful farms were found to be in locations with suitable soils and climate. A sewage farm in Pasadena, California is shown in Fig. 3 (Kinnicutt et al., 1910). Kinnicutt et al. (1910) was so enthusiastic about the future of sewage farming in the arid western U.S. that they predicted "it is likely to be the prevailing method of the future in such regions."

As predicted, water and nutrient capture and reuse is coming to the forefront in many areas in North America.

Which begs the question, is large-scale, city-wide infrastructure the most appropriate choice for the 21st century? If we build more on-site or neighborhood resource treatment systems, there is the potential for greater sustainability and infrastructure independence resulting in:

- Less intrusive land development;
- Utilization of soil for treatment;
- Reduced impact on the overall watershed;
- Greater opportunity to promote water and nutrient capture and reuse at or close to the point of origin;
- Opportunity for an integrated food, water and energy plan;
- Less use of power to move and treat resource water and potable water;
- Improved national and homeland security as a result of less reliance on centralized utilities;
- Individual, family and community buy-in and investment, and;
- Personal accountability for water resources.

CONCLUSION

Looking back at the history of effluent treatment on-site, a return to more decentralized treatment, on-site resource water treatment and water reuse systems is necessary and beneficial. We have the opportunity and the potential to save families and save the country as water becomes more precious and the economic realities of centralization continue to plague the U.S.

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Figure 1. The Perfection Septic Tank – Front Page


ADVANCE SHEET No. 5

SEWAGE DISPOSAL WITHOUT SEWERS

THE "PERFECTION" SEPTIC TANK REDUCES HOUSE SEWAGE TO WATER. NO CLEANING OUT REQUIRED. BURIED OUT OF SIGHT AND FORGOTTEN. NO FILTH. NO DISEASE GERMS.

Thoroughly Proven Through Years of Use
More Than Four Thousand Satisfied Users

GENERAL SPECIFICATIONS

CONSTRUCTION—Reinforced cement concrete. Made with grooves, easily fitted. Shipped in sections (crated). Weight, 2,800 lbs.

SIZE OF SINGLE UNIT TANK (Up to Eight People)—7 ft. 4 in. long, 32 in. wide, and 27 in. deep, inside. Walls, 2 in. thick. Size of elbows, 4 in. Elbows and farm tile necessary are not part of the standard equipment.

THESE TANKS ARE FURNISHED IN ANY SIZE TO MEET ANY REQUIREMENT—2, 3, 4, AND 5-UNIT SYSTEMS AND LARGER—LENGTH AND CAPACITIES ARE MULTIPLES OF THE SPECIFICATIONS OF A SINGLE UNIT TANK.

FOR FACTORIES—A single unit (tank) as above is rated of sufficient capacity to accommodate fifteen persons.

FOR SCHOOLS and public buildings a single unit (tank) is rated of sufficient capacity to accommodate twenty pupils.

IN SPECIAL INSTANCES, if you will refer same to us, our engineers will be pleased to furnish blue prints and other information desired.

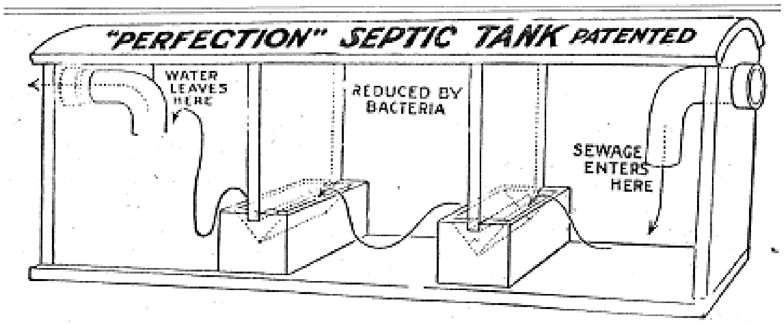
EASILY INSTALLED—Complete printed directions for setting with each order.

Estimates Furnished for Schools Factories and Public Buildings

(OVER)

Figure 2. The

ADVANCE SHEET No. 5



SEWAGE DISPOSAL WITHOUT SEWERS

EVERYTHING in nature dies and is subject to decay, from the smallest blade of grass to the highest form of animal life. This process is brought about by the action of bacteria, which are minute, living organisms, invisible to the eye, yet so numerous and so powerful as to be able to literally eat up and digest all dead matter. They are everywhere present—in water, in air, in the soil and in our bodies. Some of them are known to stimulate certain specific diseases and the destruction of these by medical treatment is known as the antiseptic process. Others, and the vast majority, are constructive and helpful in their nature and their work is known as the septic process, or the process which destroys or liquifies all dead organic matter, therefore, a spetic tank is a tank in which all house sewage or waste is reduced to liquid by this bacterial process.

Some of the filth destroying bacteria work best in the open air, others do their work only in a dark, air-tight receptacle, where the temperature does not drop below 50 degrees. The latter is the class that is formed and operates in a "Perfection" septic tank.

The temperature of sewage is an important consideration of the process of purification, therefore the life of bacteria that operate in a "Perfection" septic tank is preserved and increased in a temperature that does not fall below 50 degrees, so that it is necessary to get the tank deep enough in the ground to maintain this minimum temperature in the coldest weather. The action of the bacteria will be the same in winter and summer, as the bacteria themselves generate heat, and as more heat breeds more bacteria, and as the old bacteria are destroyed by the young, the process of decomposition is never checked. The "Perfection" septic tank, being constructed of reinforced concrete, with walls two inches thick all around, and air-tight, the contents will not be so susceptible to change of temperature as if it were made of some material that is a good conductor of heat and cold. This septic tank, or digestion tank more properly named, is simply a reservoir of three or more compartments, owing to the size needed, with a capacity equal to the average flow of say twelve hours.

The sewage entering at one end loses its velocity immediately, and its heavier suspended solids begin to sink, while the lighter rise to the top as scum. This separation of solids and liquids continues until practically all the suspended matter is eliminated. The clarified liquid then escapes gently at the far end of the tank. This action continues from day to day, the solids accumulating until those which were first deposited and attacked by the bacteria, which colonized the tank, have become liquified and pass out; thus the progress of liquidation prevents the passing out of new solids.

The sewage enters the tank at the inlet chamber and flows slowly and gently through the dividing trough into each successive chamber, from which it flows through the outlet pipe.

It takes from a week to ten days, owing to the amount of sewage, for the tank to become filled, and the bacteria to get to work, after which time the flow of water at the outlet end will equal the flow of sewage at the inlet end.

Prices and full information on application.

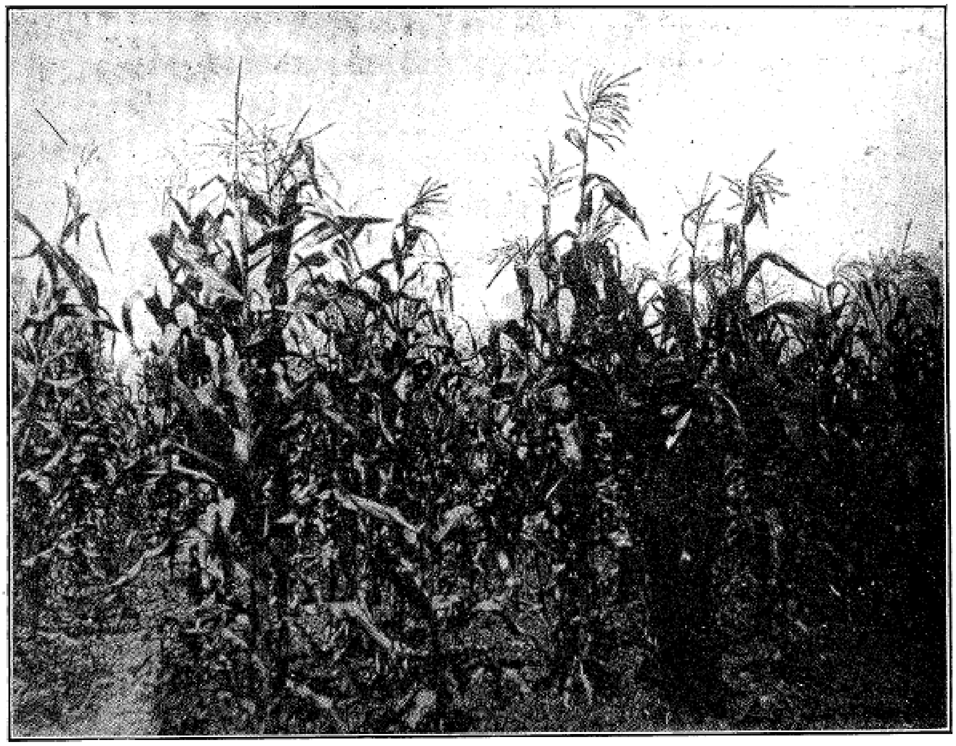
The Southern Supply Co.

WHOLESALE DISTRIBUTORS

Baltimore, Maryland

(OVER)

Figure 3. Early Sustainable Reuse – Pasadena, California Sewage Farm Cornfield



Public Confidence in Onsite Systems Requires Field Testing and Field Standards for Performance

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ABSTRACT

Although many modern, advanced systems for the treatment of onsite wastewater have passed test center standards, they struggle or fail in the field. This is due to the fact that actual onsite conditions (i.e. waste strength, wastewater temperature, daily flow patterns, etc.) are considerably different than test center conditions. NSF Standard 40 or 245 testing provides relative data based on a limited, standardized range of input characteristics over a short period of time; it does not provide data or adjustment factors relative to actual field performance capabilities. The current high rate of system failures proves that. So, while the standard may be capable of establishing a comparative benchmark between treatment systems or the ability of a system to achieve target effluent quality, a Field Verification standard is needed to establish data and information relative to the sustainability of a system under actual field conditions and its ability to substantially meet local discharge requirements. A Field Verification standard is the only way to truly enhance and complete test-center testing to reflect realistic conditions and to establish the long-term sustainability of onsite systems. Thus, test center testing would be followed by long-term jurisdictional field testing and performance audits that include 1) the sampling of key performance indicators such as turbidity, pH, DO, sludge, scum, and settle-able solids during regular service visits, in addition to verification of mechanical functionality; 2) mandatory random field audits to verify performance as a condition of maintaining approval status; and 3) online record-keeping for regulator review.

In 1970, when NSF International (NSF)/American National Standards Institute (ANSI) Standard 40 testing protocol was developed, regulators and the public were chiefly concerned with the prevention of wastewater surfacing from treatment systems. Evaluation focused on biochemical oxygen demand (BOD) and total suspended solids (TSS) removal, and, at the time, it was widely accepted that a test-center protocol was adequate for such evaluation.

However, the needs of 1970 are not the needs of today. Treatment systems are more complicated now as a result of being required to achieve greater levels of removal and to remove more constituents (ammonia, nitrate, oils and grease, coliform, etc.), while maintaining adequate pH and oxygen levels. In addition, today's regulatory jurisdictions are conscious of a larger mission. They must protect surface and ground water resources, limit nutrient inputs to sensitive ecosystems, and keep groundwater free of pharmaceuticals and pathogens, while staving off pressure from political and business entities.

Today's regulator needs to be armed with performance data that indicates how a system will perform in the field, over a long period of time, with a high level of certainty. Yet Standard 40 does not provide this data. The high rate of system failures proves that a new standard, one that involves field testing and long-term auditing, is needed (Roeder and Brookman 2006, Heufelder et al. 2007).

DIFFERENT JURISDICTIONS, DIFFERENT NEEDS

Although most U.S. states rely on Standard 40 when they approve onsite technologies, the actual requirements of each jurisdiction vary considerably. In addition to mandating different performance levels for BOD_5/TSS effluent quality, nutrient reduction, and coliform removal, jurisdictions differ greatly in their requirements for hydraulic and organic loading rates and have often identified different parameters of concern such as nitrogen, $cBOD_5/BOD_5$, or gallons per day (GPD). For example, Hawaii may need a system that produces 25/30 mg/L $cBOD_5$/TSS at 800 gpd, whereas Minnesota may want systems that can treat to 15/15 mg/L $cBOD_5$/TSS at 600 gpd. Yet both of these jurisdictions only approve or acknowledge one choice for a standardized test: NSF Standard 40. In order for a manufacturer to qualify for approval in both aforementioned jurisdictions, they would have to test through NSF 40 twice at different hydraulic loading rates, i.e., 600 gpd versus 800 gpd. This not only delays time to market but also imposes great expense on the manufacturer and ultimately the public.

Because of these variations, many jurisdictions require manufacturers to provide supplemental data and/or testing. This places the regulatory community in the awkward position of having to evaluate and compare technologies with disparate data sets and non-equivalent testing protocols. Evaluating technologies on a case-by-case basis requires jurisdictional staff to be highly trained in order to critically evaluate new technologies and their data sets. Jurisdictions typically form technical advisory committees, and manufacturers send representatives to inform the committees about their technology and explain the associated data. This can be a lengthy and expensive process, taking time that regulators could be using to carry out other critical functions of their job, such as enforcing operation and maintenance requirements and monitoring system performance.

In addition to expense and time, there is liability associated with approving technologies that are difficult to compare and have widely varying data on which an approval will be based. Regulatory bodies are often in the difficult position of defending their decision both to other manufacturers and to their supervising body. They may and have in fact already found themselves defending their choices in court. This does nothing but increase the divide between the manufacturing and regulatory communities, with public health and the environment left somewhere in the middle. As a result, it is understandable that the vast majority of jurisdictions are reluctant if not totally unwilling to take this approach. This leaves Standard 40 as the most widely accepted standard protocol that jurisdictions can use to reduce some of the liability associated with approving new technologies in their jurisdictions.

THE LIMITATIONS OF STANDARD 40

A small number of jurisdictions seek to reduce this liability even further by mandating Standard 40 Certification. Certification by NSF ensures several things. First, NSF will audit the manufacturing facilities to ensure quality of production. Second, NSF will ensure that all parts and pieces that comprise the Certified model are identical to those that are being installed in the field. Third, NSF will audit 12 systems each year in the field to ensure that they are the same as the Certified model.

The current certification program also mandates that a two-year service contract be included in the price of a system when it is initially sold. However, there is no requirement (other than to offer an extended service contract) for service or auditing after the initial two-year period.

Historically, there has been no provision for auditing system performance in the field, such as effluent testing. To address the concerns regarding field performance verification, NSF has recently adopted a protocol under NSF/ANSI 360-2010 for a field-testing program, but it is not being widely used. The 360 standard could be adapted, or it could be used as a template, to establish a program suitable for local jurisdictional needs.

The NSF certification program was initiated to address needs expressed by stakeholders in the industry, including regulatory jurisdictions that wanted a better idea of how systems in the ground were being serviced and how they were functioning. In theory, this program was a great idea. But in practice, it lacks the robustness to provide jurisdictions with any meaningful information about systems in their respective jurisdictions. Because only 12 systems per manufacturer are audited each year nationwide, several years may pass between NSF audits of a particular system in any one jurisdiction. For these reasons, it would be prudent for jurisdictions to perform their own audits of approved technologies and use the results to maintain approval status.

In addition to the problems created by the lack of field testing, the current Standard 40 testing protocols themselves create other problems. These fall into two categories: problems with respect to BOD/TSS removal, and issues that the current standard does not address at all, such as variations in hydraulic and organic loading, Total Nitrogen reduction, coliform reduction, cold weather performance, service intervals, and reliability of the system components.

Below are some of the current limitations of Standard 40:

- Bench tests utilize an “idealized” influent waste stream. For example, Standard 40 allows influent levels as low as 1/3 the concentration of typical residential wastewater per Crites and Tchobanoglous (1998).

- Idealized temperature conditions are chosen by manufacturers. Test centers are located in different geographic regions of the country with different climates. Temperature affects a system’s ability to nitrify; thus, performance can be over- or underrepresented depending on the testing location.

- Test center “stress periods” often don’t realistically simulate real stresses. Samples are not collected until 24-48 hours after the stress period.

- Test duration is too short -- only six months. Performance issues may not become apparent in the first six months of operation.

Reliance on such a standard sends the message to manufacturers that they need only create a system that can perform for six months and only to minimum standards. Consequently, there are some technologies that pass Standard 40, and cannot maintain that same level of performance in the field. This leads to contentiousness between the regulatory and manufacturing sectors, and citizens are left holding the bag when their approved systems fail and they must repair or replace them. Simply put, reliance on Standard 40 is not enough. We need to encourage enforceable jurisdictional field audits as part of any approval process.

Should a regulatory jurisdiction decide to require information on nutrient reduction, coliform removal, or increased hydraulic/organic loading rates, manufacturers have been required to test under NSF 245 (which has similar limitations to Standard 40, if not more) for TN reduction, create and execute a custom disinfection test, and retest their system under Standard 40 again with different loading conditions. Not only is this extremely expensive, but it is also time-consuming and leaves many performance questions unanswered. Without these answers, regulatory jurisdictions are left in the same place they began -- wanting to approve technologies suitable to protect the environment and public health, but lacking the proper data to make such decisions.

If the public or regulators lose confidence in onsite wastewater treatment, governments will start making unwise and expensive decisions, such as imposing construction moratoria and requiring centralized sewer systems where they are inefficient. So what can the onsite/decentralized industry do to help create a higher level of confidence in advanced onsite treatment technology? The answer is simple: revamp Standard 40 to require field-testing, enforce field operation and performance via jurisdictional audits to meet today's regulatory needs, and establish a Field Verification Protocol.

It is important to pause and emphasize that this paper is not intended as an admonishment of NSF. Rather, it is an overdue call to action for manufacturers, regulatory bodies, testing organizations—and to all stakeholders in this industry—to come together and address the issues that face us and, in fact, threaten the integrity of our industry.

THE ELEMENTS OF A GOOD STANDARD

So, what would an ideal standard for approving and verifying field performance of onsite systems be like? First, the Standard 40 test-center component should be revised to address the limitations mentioned above; for example, the requirements for influent quality and stress periods should be revised to create a more realistic challenge. The information available to regulators should include a full report of performance, including all testing results that are to be used to gain approval, all mechanical/electrical issues experienced during the test, and the operating cost of each technology tested. Second, there should be field testing and ongoing performance audits of systems in operation. This could be implemented as part of Standard 40, or it could be done in each jurisdiction. A performance audit program should include the following elements:

• Sampling of turbidity, pH, DO, sludge, scum, and settle-able solids, during scheduled service visits, as field performance indicators, with the results electronically submitted to the jurisdiction's database. As well, the service provider should record the condition of the observable parts and pieces of the system, to provide some idea of the system's mechanical durability and life-cycle costs. If one of these samples is outside of a mandated threshold, further sampling and analysis should be required, along with repair or alteration to the system to ensure compliance.

• Mandatory random field audits to maintain approval status. This should be used as an enforcement mechanism after systems have passed a field and/or test center test. Jurisdictions should audit some percentage of installed systems annually, requiring a certain percentage of systems to be in compliance with the standard under which they were approved. If manufacturers are out of compliance, their approval status should be suspended until further evaluations can be conducted.

• Online record-keeping that makes records of service visits, sample results, and the status of O&M contracts available for regulators to view at any time.

Once a field-testing protocol was developed, local jurisdictions could adapt it to their own needs, taking local soils and climate into consideration. This would allow memoranda of understanding between jurisdictions that have similar soils, climates, and regulatory requirements, eliminating redundant testing for the most part. Qualified local laboratories would carry out sample analysis, reducing overall program costs. These costs would be borne by the manufacturer being tested.

WHY A NEW STANDARD CAN SUCCEED NOW

Some readers may think to themselves that this effort is doomed for two reasons. For one thing, efforts to address these issues have so far met with little success. For another thing, the task of collecting and managing performance data seems daunting. However, if stakeholders commit themselves now to the idea of field testing, there are currently two factors that will contribute to its success.

The first factor is that the technology to solve the data-collection and data-management problems is readily available right now. Several groups have developed open-source databases that can receive information from many different manufacturers and record it in a searchable database. A regulatory body could know which systems had been serviced, find out which were under a service contract, and view information about the performance of each system. Moreover, field performance of onsite systems can be evaluated through field indicators that the service provider can easily and inexpensively measure during a service visit. These could include measurements of temperature, dissolved oxygen (DO), turbidity, and pH (ammonium, nitrate, alkalinity, chlorides, etc., as necessary). The service provider can also verify whether mechanical components are working, such as blowers, floats, and UV or chlorine disinfection systems. With this relatively inexpensive technology, regulatory jurisdictions could implement their own jurisdictional audit programs.

The second factor is the wide and fast-growing public support for better stewardship of the environment. While this is not true in every part of the country, it is certainly a growing movement, and one that people are willing to spend money to support.

Nevertheless, as we move into an era when environmental controls will be more stringent, and we ask people to spend more and more money to treat their wastewater to higher and higher levels, both government and industry will need to be able to demonstrate that this is money and time well spent. A jurisdictional audit program would provide the industry with some long-needed credibility with our customer base. We as an industry and as a government can say, “Yes, we do care about the expense we are requiring, and yes, we do care about the quality of the environment we are charged with protecting. Here is the data to show that we are doing the best job we can with the available technologies”.

A CHANGE THAT WILL BENEFIT EVERYONE

Many advanced treatment systems that have passed test center testing are failing in the field. These failures frustrate homeowners, endanger public health, and give the onsite treatment industry a black eye.

Decision makers and stakeholders should formulate a field testing protocol, which jurisdictions could adapt to their own requirements. A robust field test standard would allow governments and homeowners to feel confident about relying on advanced onsite treatment. Orenco and other manufacturers will benefit greatly from this, and we look forward to cooperating with other stakeholders on such an initiative. We hope that other manufacturers, state wastewater organizations, and regulatory bodies will recognize the value of field testing and take a leadership role in developing this new approach.

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The Centralized Myth - Soil to the Rescue.

Dennis Hallahan, Infiltrator Systems Inc.

ABSTRACT

SSSA Onsite Wastewater Conference, Innovation in Soil-Based Onsite Wastewater Treatment April 7-8, 2014, Albuquerque, NM Type of Abstract: Community Systems Title of Paper: The Centralized Myth - Soil to the Rescue Abstract In most cases perception is reality, such is the case in the wastewater world. The centralized model has been perceived to be the Cadillac, the approach of choice, the most effective method for the long haul. This while the decentralized model has clawed its way from “ugly stepchild” to alternative, advanced, and cost effective wastewater treatment solution. Cities and towns throughout North America are facing the challenge of dealing with large volumes of wastewater discharged, often with minimal treatment, into concentrated locations such as rivers and other natural waterways. Centralized sewers, often seen as the preferred solution to wastewater issues, are a primary contributor to the discharge to surface waters problem. With a sustainable decentralized approach to wastewater treatment groundwater is extracted, consumed, treated onsite and close to its point of origin to recharge the aquifer. Decentralized systems can treat to the same level as centralized systems. Whatever size flow the project has, whether it is 5,000 gallons per day (gpd), 500,000 gpd, and even over 1 MGD a treatment solution using the decentralized model is available. This presentation will review the centralized model and its shortcomings and then discuss the overall benefits of the decentralized model and then review some large system case studies thus overturning the long held and common misperceptions. 6. About the Presenter Dennis F. Hallahan, PE Mr. Hallahan has over twenty years of experience with onsite wastewater treatment systems’ design and construction. He has authored several articles for onsite industry magazines and has given numerous presentations nationally on the science and fundamentals of onsite wastewater treatment systems. Dennis is currently Technical Director at Infiltrator Systems, where he is responsible for government relations and technology transfer between Infiltrator Systems and the regulatory and design communities. Dennis also oversees a staff that is responsible for product research and testing for both universities and private consultants. He received his MS in civil engineering from the University of Connecticut and his BS in civil engineering from the University of Vermont. Dennis is a registered professional engineer in Connecticut. Dennis also holds several patents for on-site wastewater products. Contact info: Dennis Hallahan 4 Business Park Rd. Old Saybrook, CT 06475 P: 800.221.4436 dhallahan@infiltratorsystems.net.