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**2006
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WATER SCIENCE
AND
TECHNOLOGY
SYMPOSIUM**



Virginia
WATER RESOURCES
Research Center

PROCEEDINGS



**VIRGINIA POLYTECHNIC INSTITUTE AND STATE UNIVERSITY
BLACKSBURG, VIRGINIA**

2008

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PROCEEDINGS

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DEDICATION

The Proceedings of the 2006 Virginia Water Science and Technology Symposium are dedicated to the victims of the April 16, 2007 events on the Virginia Tech campus, particularly in memory of the following water scientists:

Brian Roy Bluhm, Civil and Environmental Engineering, Master's student
Matthew Gregory Gwaltney, Civil and Environmental Engineering, Master's student
Jeremy Michael Herbstritt, Civil and Environmental Engineering, Master's student
Jarrett Lee Lane, Civil and Environmental Engineering, Senior
G.V. Loganathan, Civil and Environmental Engineering, Professor
Partahi M. "Mora" Lumbantoruan, Civil and Environmental Engineering, Ph.D. student
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Juan Ramón Ortiz-Ortiz, Civil and Environmental Engineering, Master's student
Julia Kathleen Pryde, Biological Systems Engineering, Master's student
Waleed Mohamed Shaalan, Civil and Environmental Engineering, Master's student

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MANAGING RISKS TO VULNERABLE WATER RESOURCES SYSTEMS THROUGH PREPAREDNESS AND RESILIENCE

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ABSTRACT

To develop and successfully deploy a risk-based national comprehensive enterprise approach to preparedness for natural hazards and terrorist attacks, we must understand and appreciate the intricate relationship between preparedness and resilience and their centrality in managing risks to our water resources systems. Are water supply systems relatively more vulnerable to terrorist threat or natural hazards or accidents than other infrastructures? As the operations of water supply systems become more automated through supervisory control and data acquisition (SCADA) systems, they will be more open to remote threats through means of information warfare.

The vulnerability of the nation's water supply infrastructure from major natural hazards and/or acts of terrorism should be viewed as a long-term endeavor. The nature and degree of vulnerability of the water infrastructure will change with time as the nation's population and economy shift geographically, as the climate changes, and as the motivations and ability of terrorists to carry out threats against water supply systems grow.

This talk will address these issues in the context of the principles that guide risk assessment and management, the definition of vulnerability in terms of the states of the water system, and the characterization of the interdependencies among water resources systems and other critical infrastructures.

USING SYSTEM DYNAMICS MODELING TO UNDERSTAND THE HYDRAULIC GEOMETRY OF FOREST STREAMS

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KEYWORDS: modeling, feedback, system dynamics, stream flow, fluvial geomorphology

ABSTRACT

Streams experience a wide range of water flows. As water moves through a stream reach from the channel upstream, the observed velocity, depth, and width of flow reflect the hydraulic characteristics of the channel cross section. Graphs of these parameters as functions of discharge constitute the hydraulic geometry of a stream channel (Leopold, Wolman, Miller 1964).

A distinction may be drawn between *flow geometry*, which includes interaction among the three flow variables, and *channel geometry*, which refers to the three-dimensional form of the channel fashioned by water movement to accommodate discharge and sediment load. The two are related but not necessarily in a simple way (Knighton 1998).

This paper highlights a simulation model designed to investigate this interaction. The dynamic nature of hydraulic geometry and its interconnectedness with channel geometry and sediment movement is explored. Feedback links are identified and simulations reveal the consequences of changing one or more channel or flow parameters. Results are displayed in diagrams and graphs. The model may be used as an effective tool for making comparisons among stream management scenarios and proposed land use policies.

INTRODUCTION

As water flows it interacts with land, forming a channel. This interaction creates a *flow geometry* and *channel geometry* unique to the particulars of landform and energy gradient. As water in a stream system moves through the landscape the potential energy of elevation is changed to the kinetic energy of motion. The interactions among feedback processes associated with water flow and surface friction cause land surface and water column to together shape the streambed. The streambed shape helps determine the efficiency of water movement and rate of sediment production.

Measurements of water flow that characterize water width, water depth, and water velocity as functions of water discharge may be used to determine the overall hydraulic geometry of a stream over a range of water discharge rates. This hydraulic geometry may be organized by Rosgen Stream Class to further understand the unique attributes of channel flow and links to channel geometry. With empirical measurements of hydraulic geometry in hand and organized

by Rosgen Stream Class, predictions may be made about the shape and velocity of stream flow, and efficiencies and rates of sediment production.

System dynamics modeling can be used to accurately trace interconnections and feedbacks in complex dynamic systems. With respect to streams and landform, this means that interactions associated with hydraulic regime and channel geometry may be modeled. Coefficients of hydraulic geometry may be used as model parameters. Comparisons of water velocities, shear velocities, and water discharge rates may be made among Rosgen Stream Class types of known hydraulic geometry. The dynamics of water movement and channel shape may be investigated and interactions among hydraulic geometry and channel form in new and variant channel types may be tested.

This paper highlights the potential use of system dynamics modeling as an aid to understanding the hydraulic geometry and channel form of forest streams. Measurements of hydraulic geometry from forest reference streams of differing Rosgen Stream Classes are used as parameters in a system dynamics model. The model may be used to compare water velocities and shear velocities among differing Rosgen Stream Classes. These values may be used to help determine the potential sediment loads associated with various stream types.

METHODS

A predicted hydraulic geometry for each of five forest reference streams was determined from non-linear regression of detailed at-a-station field measurements of water width, depth, velocity and discharge collected over an eight-year interval. The stream reach at each measurement site was classified using the Rosgen Stream Classification system (Catena 1995).

A system dynamics simulation model was constructed describing the interconnections among rainfall, water discharge, water surface slope, water velocity, channel roughness, cross-sectional area, mean water depth, water width, wetted perimeter, and hydraulic radius in a forested watershed. Equations were written describing each link in the system. The empirically derived equations describing hydraulic geometry as indexed to bankfull discharge were used as parameters in the model (see Figure 1). A computer was used to process the equations. Simulated outcomes showing water velocities and shear velocities of various Rosgen Stream Classes were displayed and compared as graphs of change over time.

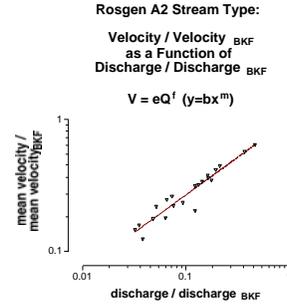
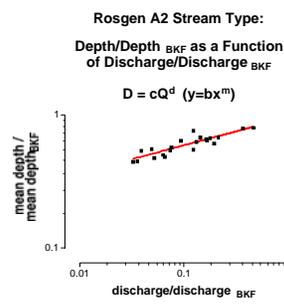
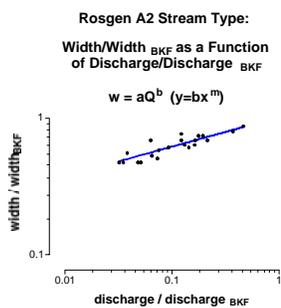
VDOF Rosgen A2 Forest Reference Stream: Measured Hydraulic Geometry: Bankfull Indices

$$a \times c \times e = 0.99 \quad b + d + f = 1.00$$

Best-fit values	
B	1.024
M	0.2161
Std. Error	
B	0.05848
M	0.02636
95% Confidence Intervals	
B	0.9016 to 1.146
M	0.1609 to 0.2712
Goodness of Fit	
Degrees of Freedom	19
R squared	0.7778
Absolute Sum of Squares	0.06149
Sy.x	0.05689
Runs test	
Points above curve	10
Points below curve	11
Number of runs?? [X not in order]	11
P value (runs test)	
Deviation from Model	
Data	
Number of X values	21
Number of Y replicates	1
Total number of values	21
Number of missing values	0

Best-fit values	
B	0.9627
M	0.2082
Std. Error	
B	0.05080
M	0.02427
95% Confidence Intervals	
B	0.8563 to 1.069
M	0.1574 to 0.2590
Goodness of Fit	
Degrees of Freedom	19
R squared	0.7936
Absolute Sum of Squares	0.04759
Sy.x	0.05005
Runs test	
Points above curve	10
Points below curve	11
Number of runs?? [X not in order]	11
P value (runs test)	
Deviation from Model	
Data	
Number of X values	21
Number of Y replicates	1
Total number of values	21
Number of missing values	0

Best-fit values	
B	1.009
M	0.5739
Std. Error	
B	0.05545
M	0.03040
95% Confidence Intervals	
B	0.8925 to 1.125
M	0.5102 to 0.6375
Goodness of Fit	
Degrees of Freedom	19
R squared	0.9496
Absolute Sum of Squares	0.01887
Sy.x	0.03151
Runs test	
Points above curve	13
Points below curve	8
Number of runs?? [X not in order]	12
P value (runs test)	
Deviation from Model	
Data	
Number of X values	21
Number of Y replicates	1
Total number of values	21
Number of missing values	0



Samuel H. Austin: July 15, 2003

Figure 1: An Example of Hydraulic Geometry Measurements Indexed to Bankfull Discharge

RESULTS

Comparisons of predicted shear velocity, water velocity, and the flow determined eighty-fourth percentile streambed particle sizes among Rosgen Stream Classes, help illustrate functional differences among channel types.

Two examples of this comparison are shown below. In the first comparison, the model illustrates the differences in water velocity and shear velocity as a Rosgen Class A2 forest stream is compared with a Rosgen Class C4 forest stream.

In the second comparison a scenario analysis is performed. The model illustrates the relative changes in water velocity and shear velocity that may occur in each stream if a section of each channel were shortened (channelized) to one-half of its previous length.

Comparison One

Figure 2 illustrates a predicted shear velocity, water velocity and D84 particle size for a Rosgen Class A2 forest stream with an average water surface slope of 0.05, as it responds to a 24-hour 2.5-inch rainfall. Notice that shear velocity and water velocity increase as the storm progresses. Increased water flow intercepts larger bed material on average, causing shear velocity and predicted average D84 particle size to increase and then decrease slightly with water flow. Stream water velocity increases but does not surpass predicted average values of shear velocity.

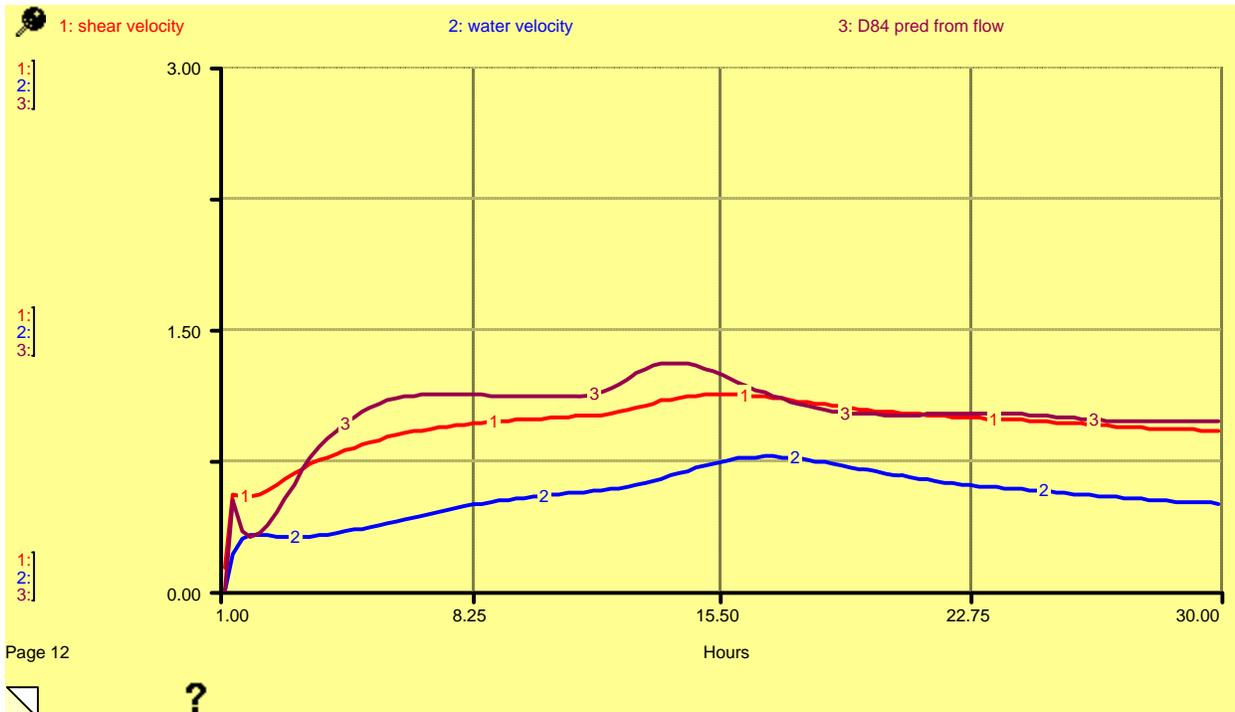


Figure 2: Predicted Shear Velocity, Water Velocity and D84 Particle Size for a Rosgen Class A2 Forest Stream With An Average Water Surface Slope of 0.05, After A 2.5-Inch Rainfall.

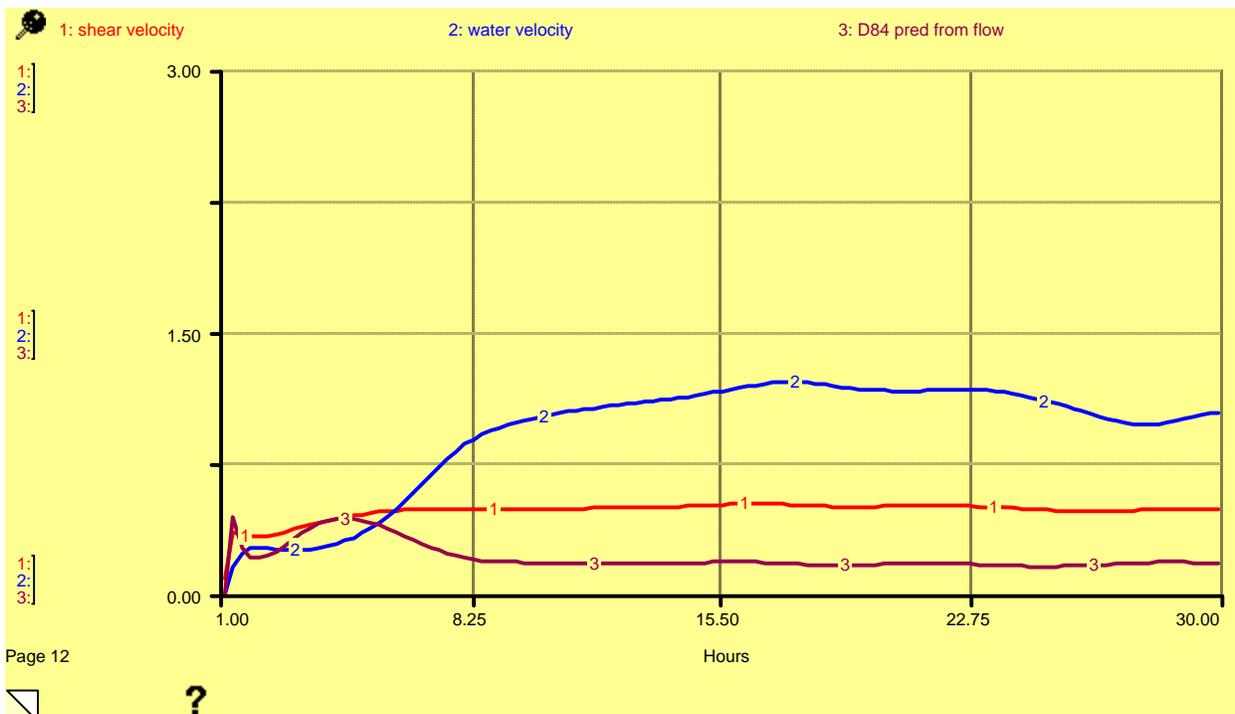


Figure 3: Predicted Shear Velocity, Water Velocity and D84 Particle Size for a Rosgen Class C4 Forest Stream With An Average Water Surface Slope of 0.02, After A 2.5-Inch Rainfall.

Figure 3 illustrates a predicted shear velocity, water velocity and D84 particle size for a Rosgen Class C4 forest stream with an average water surface slope of 0.02, as it responds to a similar 24-hour 2.5-inch rainfall. Notice that the stream response is somewhat different. Shear velocity increases slightly as the storm is initiated, but water velocity quickly exceeds predicted shear velocities. Increased water flow intercepts smaller bed material on average, causing shear velocity and predicted average D84 particle size to increase slightly and then decrease with water flow. Predicted shear velocities stay relatively uniform and predicted velocities surpass them.

Comparison Two

Figure 4 illustrates a predicted shear velocity, water velocity and D84 particle size for a Rosgen Class A2 forest stream that has been shortened (channelized) with an average new water surface slope of 0.10, as it responds to a 24-hour 2.5-inch rainfall. It is interesting to compare Figure 4 with Figure 2. Notice that in the channelized A2 stream water velocity and shear velocity both increase compared to the non-channelized Class A2 stream, while predicted average D84 particle size remains essentially the same. This suggests that the new slope regime in the A2 channel does modify the dynamics of shear velocity and water velocity producing more aggressive water movement. In this instance the large bed particle sizes and generally robust channel form associated with the A2 channel geometry help prevent water velocities from exceeding shear velocities.

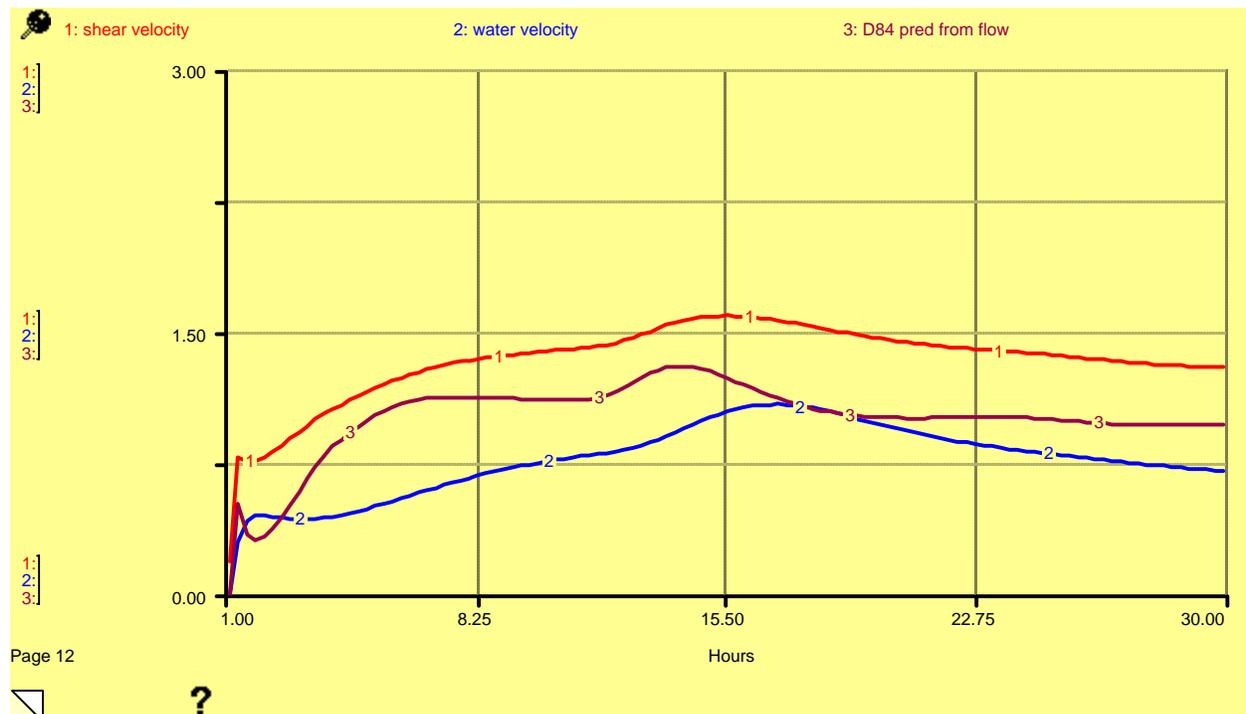


Figure 4: Predicted Shear Velocity, Water Velocity and D84 Particle Size for a Shortened Rosgen Class A2 Forest Stream With An Average Water Surface Slope of 0.10, After A 2.5-Inch Rainfall.

Figure 5 illustrates a predicted shear velocity, water velocity and D84 particle size for a Rosgen Class C4 forest stream that has been shortened (channelized) with an average new water surface

slope of 0.04, as it responds to a 24-hour 2.5-inch rainfall. It is interesting to compare Figure 5 with Figure 3. Notice that in the channelized C4 stream water velocity increases significantly and shear velocity increases slightly as compared to the non-channelized Class C4 stream. Predicted average D84 particle size remains essentially the same. This suggests that the new, steeper, slope regime in the C4 channel significantly changes the dynamics of shear and water velocity. More aggressive water movement appears to be produced, with potential increases in channel scour and sediment volumes. In this instance the increasingly smaller bed and near bank particle sizes contacted by larger water flows help dampen C4 shear velocities and predicted average D84 particle sizes ensuring ample opportunity for movement of channel material and potentially increased sediment production.

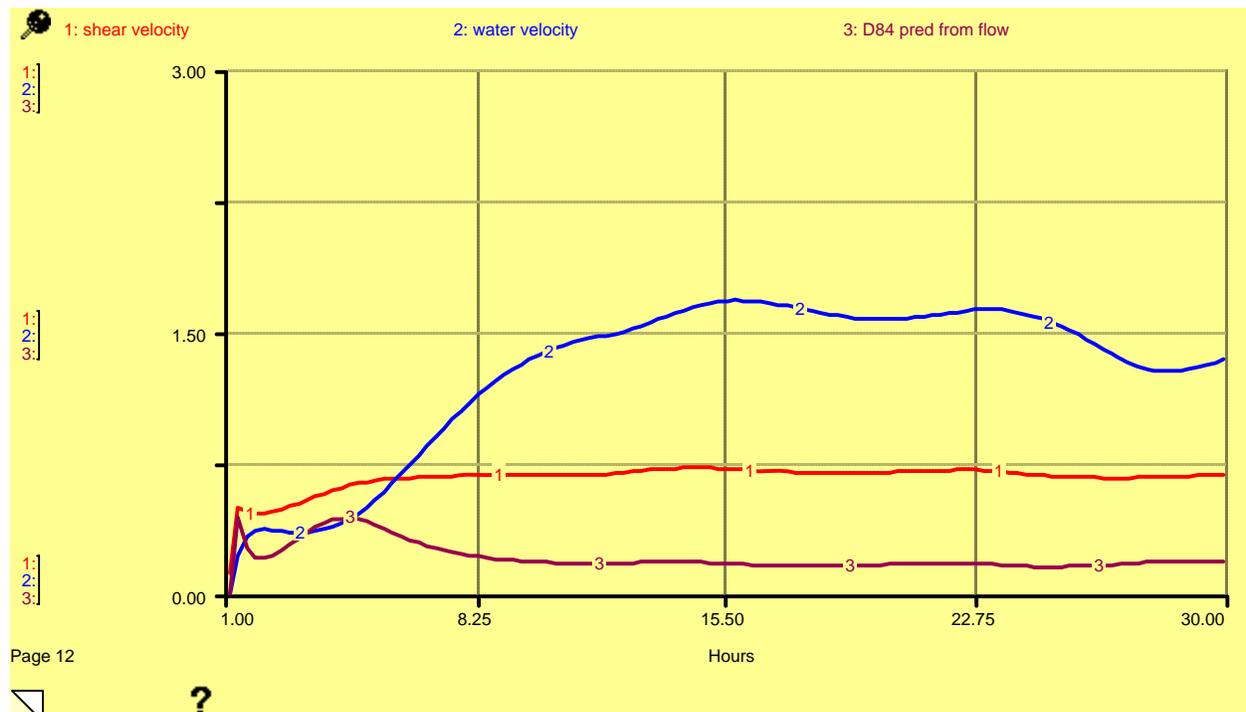


Figure 5: Predicted Shear Velocity, Water Velocity and D84 Particle Size for a Shortened Rosgen Class C4 Forest Stream With An Average Water Surface Slope of 0.04, After A 2.5-Inch Rainfall.

DISCUSSION

Many facets of interconnectedness and dynamics associated with the interaction between flow geometry and channel geometry may be investigated using empirically derived hydraulic geometry relationships combined with system dynamics modeling. Water energy slope, particle shear stresses, shear velocity, water velocity and stream flow are all part of a connected feedback system. Variables such as rainfall, water discharge, water surface slope, water velocity, channel roughness, cross-sectional area, mean water depth, water width, wetted perimeter, and hydraulic radius play key roles in the dynamics of channel systems. Seemingly small changes in one or more variables can set reinforcing or compensating processes in motion that have the potential to move through the system and influence other parameters. This idea is illustrated by positive feedback processes associated with changing water velocity and channel slope, and

compensating process associated with bed particle size and energy dissipation. Much may be learned from the interplay among variables. Hopefully this discussion has illuminated a small sampling of the richly interwoven dynamics associated with the interaction of water flow and channel geometry.

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PROBABILISTIC MONITORING IN VIRGINIA: EXPERIENCES FROM THE FIRST FIVE YEARS

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In 2001, the Virginia Department of Environmental Quality (VDEQ) started a 5 year project to assess the utility of a probability-based monitoring program (ProbMon) as an addition to existing targeted and watershed based water quality monitoring programs. One reason for initiating ProbMon was to determine the extent of water quality problems with statistical accuracy. The program design was based on EPA's Environmental Monitoring and Assessment Program (EMAP). A major factor in our decision to use an EMAP design was the support we received from EPA, Corvallis and EPA Region III in the study design, station selection, and training. VDEQ has adapted the study design to fit specific needs by altering how stations are chosen, adapting testing methods and expanding the parameters sampled. One of the primary goals was to comprehensively assess each site. This goal resulted in the sampling of a variety of chemical, physical habitat and biological parameters and GIS based land use analysis. The data have proven useful for statewide and regional assessments, assessing the effect of new criteria, developing and testing of biomonitoring methods, prioritizing problems, and assessing the effectiveness of our programs. The design allows DEQ to sample parameters that are expensive to analyze at a limited number of sites and still achieve a meaningful statewide assessment. Virginia has decided to continue its probabilistic program for another five years.

**FACTORS INFLUENCING THE RELATIONSHIP BETWEEN TURBIDITY AND
SUSPENDED SEDIMENT CONCENTRATION:
CAN THE APPROACH BE IMPROVED?**

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KEY WORDS: sensor technologies

ABSTRACT

Suspended sediments are major water pollutants in Virginia and throughout the world. Difficulties associated with accurately quantifying sediment transport have led to recent monitoring advances, including the use of turbidity as a surrogate for estimating suspended sediment concentrations (SSC). Turbidity can be monitored remotely and continuously, and turbidity-based SSC-monitoring can be less expensive than traditional sediment-monitoring methods. Turbidity, however, is not a perfect surrogate for SSC. Variability in the turbidity-SSC relationship is influenced by the characteristics of transported sediments, as well as spatial variations in sediment transport within the stream channel. These influencing factors, in turn, may be controlled by hydrologic factors such as hydrograph characteristics during storm events, source area contributions to SSC, and/or seasonal variations among sediment sources. This presentation will discuss how these factors affect the turbidity-SSC relationship, and the potential for improving the turbidity-SSC estimation method by including these variables in the modeling approach. Preliminary data from Virginia locations regarding the application of these variables to improving turbidity-SSC estimation methods will be presented.

EXAMINING TEMPORAL CHANGES IN SOIL MOISTURE IN A KARST SINKHOLE USING DIFFERENTIAL ERT AND TDR

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ABSTRACT

At an instrumented sinkhole field site in southwestern Virginia, we are using Differential Electrical Resistivity Tomography (DERT) to monitor changes in resistivity values in sinkholes over time and space. Calibrated Time Domain Reflectometry (TDR) readings are also being used to measure soil moisture values over time. Preliminary results show that DERT can clearly model variations in soil resistivity due to slow summer drying events as well as rapid infiltration during recharge events. TDR readings show similar patterns of variation. Current work is focusing on correlating calibrated one-dimensional TDR moisture data and physical soil parameters with corresponding 2-D ERT data. If this calibration is successful, we will be able to create 2-D models of soil moisture which, when analyzed over time, will support building a model of infiltration volumes and rates and recharge to the underlying aquifer.

GROUNDWATER ASSESSMENT IN THE IVY SECTION OF ALBEMARLE COUNTY

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KEY WORDS: groundwater assessment, GIS, rural development, collaborative decision making

ABSTRACT

As major cities, such as Charlottesville, develop, they spread into the rural areas of surrounding counties. The result of this urban sprawl is increasing housing densities, groundwater consumption rates, and likelihood of contamination from septic drainfields in rural areas without public water or sewer services. County decision-makers, who grant permits for new home sites, lack access to groundwater assessment (GWA) information. In order to plan for development that can be sustained by its groundwater capacity, county decision-makers require regional GWA information. However, the fractured igneous and metamorphic bedrock formation of Albemarle County does not permit regional GWA forecasting. In this study, a model has been developed to test a representative sample of private wells, and to present the data using maps created with ArcGIS software for Internet publication.

INTRODUCTION

Many Americans, especially those in metropolitan areas, take the availability and abundance of good water for granted. For many rural people, however, adequate and safe water supplies are not guaranteed. Approximately one sixth of all Americans rely on groundwater supply that is continually threatened by increased demand and contamination of sources. A team of six Systems Engineering students under the direction of Professor Garrick E. Louis, developed a model for ground water assessment (GWA) that could be used for planning development in rural areas that depend primarily on groundwater. The project was carried out on private wells in the Ivy subdivision of Albemarle County, Virginia.

METHODS

Hydrogeology of Ivy and the Necessity of the Project

Groundwater contamination from septic systems, pavements and lawn fertilization runoffs, are growing more common as development continues in rural areas. To protect rural residents' health existing groundwater assessment methods must be examined and new ones developed. This is especially important in rural areas such as Ivy where about up to 75% of households depend entirely on private wells for drinking water (GWPS 2004). The Ivy area is dominated by water-bearing geological formations of igneous and metamorphic rock, in which water flows

through random cracks and fissures (Mudd and Daniel 2002). These fractures occur mainly close to the surface and are the only means of water transport in this type of rock (Figure 1). Because groundwater assessment in this geological formation is difficult, little is known about groundwater quality and availability in this formation (Smith 2005). Dr. Thomas Burbey, an assistant professor of geology at Virginia Tech, attributed this lack of research to the prevailing opinion that Virginia is a groundwater rich state (Mudd and Daniel 2002). This formation is unique to a few areas on the east and west coasts of the United States (Figure 2).

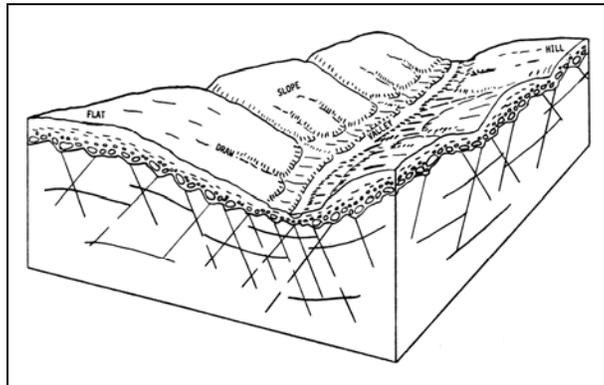


Figure 1. Fractured Metamorphic and Igneous Rock [10]

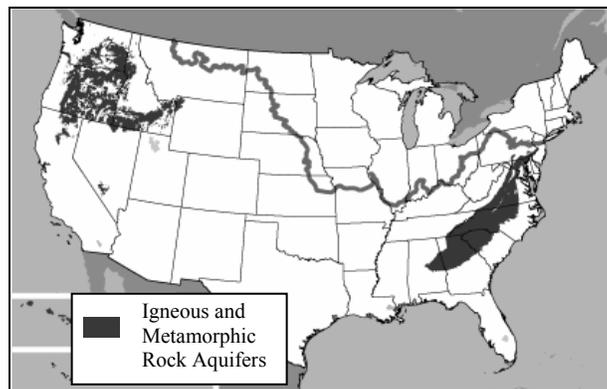


Figure 2. Igneous and Metamorphic Rock Aquifers [17]

Water in metamorphic and igneous rock is found only in openings caused by fracturing, faulting, or weathering (Feetter and Raflo 2002, p. 319-20). The combination of saprolite and incomplete weathering in the transition zone make this aquifer highly permeable and thus, susceptible to contamination (Harned and Daniel 1989). The yields of wells in this region depend on the spacing and size of water bearing fractures, the efficiency of the hydraulic connections between fractures, and the transition zone which holds most of the water (Heath 1989).

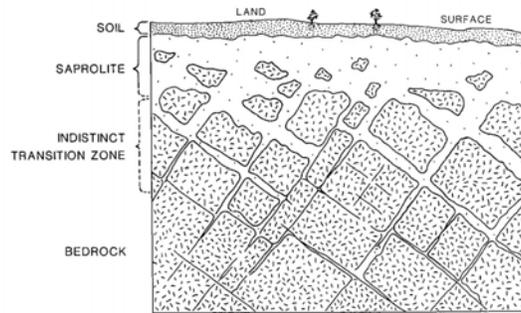


Figure 3. Fractured Igneous and Metamorphic Rock [10]

In this type of ground water system, the direction and rate of water flow are unpredictable so that the use of small scale sampling and the geo-statistical method of Kriging (Mudd and Raflo 2002) are not applicable. Freeze (1979) also notes the challenge of modeling groundwater flow in such formations (Freeze and Cherry 1979, p. 73). Despite the geological obstacles, planners need a method of groundwater assessment, the best of which is to test all the wells in the area. For this method to be both economically feasible and acceptable to the well owners, the water testing must be quick, simple, accurate, and minimally invasive. The study on Ivy can then be used as a reference for the water quality in the region.

Test Kit and Sampling Methodology

GWA conducted by the Capstone group required a methodology for selecting homes and obtaining permission from well owners. After permission letters were obtained, 100 homes were sampled. Another requirement for the GWA was a test kit capable of quick and minimally invasive tests in order to sample as many wells as possible in a short amount of time. In addition, the test kit also needs to be simple so that lay persons outside of the Capstone group can use it without much difficulty.

A previous project had developed a prototype test kit, but it was unstable. Therefore, the Capstone team redesigned the test kit to consist of a laptop for ease of use, off-the-shelf nitrate and pH test probes, compatible USB interfaces for the probes, software designed to work with the probes, and a handheld GPS receiver (Figure 4). The laptop-based kit can be expanded to accommodate probes for coliform and arsenic that may be developed in the future. The kit also includes a user's guide and instructional video.

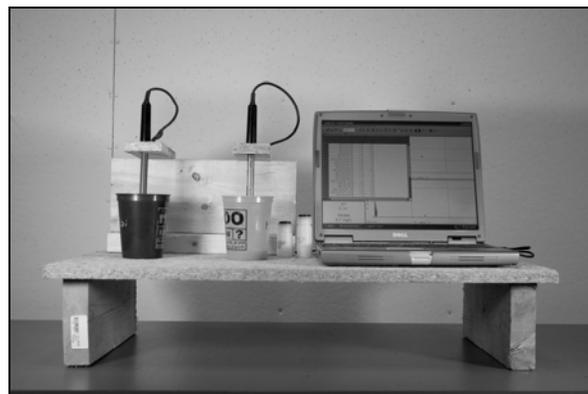


Figure 4. Redesigned Test Kit

Water Perceptions Survey

Homeowners were asked to rate the following four items on a scale of one to five, with five being the best: their water quality, the importance of their water quality, their water quantity, and the importance of their water quantity. In the two figures below, the dark bars represent well owners and the light bars represent residents relying on public water.

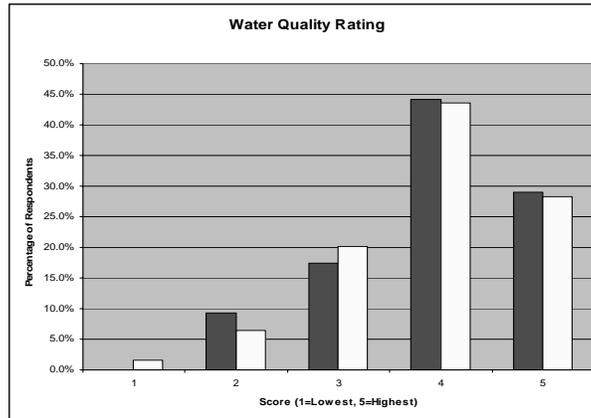


Figure 5. Water Quality Ratings

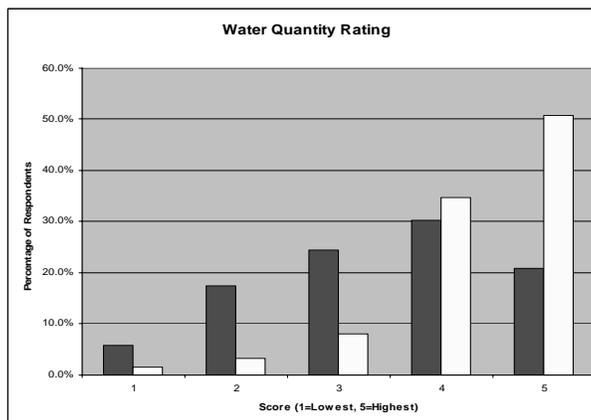


Figure 6. Water Quantity Ratings

GIS Mapping

In the past ten years, the need for a comprehensive system of private well data has been emphasized not only for the state of Virginia, but for the whole United States. The most common way to create this system is using Geographic Information Systems (GIS) software. GIS gained popularity after it became a computer based program. The team used ArcGIS to create the GWA maps out of the well sampling and survey database. ArcGIS's manufacturer, ESRI, defines ArcGIS as "integrated collection of GIS software products for building a complete GIS for your organization. The ArcGIS framework enables you to deploy GIS functionality and business logic wherever it is needed. This architecture, coupled with the geo-database, gives you the tools to assemble intelligent geographic information systems" (ESRI 2005). GIS allows users to transfer a database into the map based on coordinates in the database

Arsenic Detection in Water

Arsenic is a naturally occurring element largely found in the earth's crust, making up 0.00005% of soil, rocks, and minerals [2]. Natural and anthropogenic activities cause it to be present in groundwater. Natural forces, such as weathering, decomposition, and erosion into bodies of water, along with the decay of plants are a few of arsenic's chief sources (Chapel *et al.* 1994). Dermal exposure and ingestion of arsenic are the main pathways for its adverse human health effects. Arsenic can be lethal in large doses in a short-term period, and a long-term exposure to small doses may cause cancer and other potentially fatal health risks. For these reasons, an evaluation of arsenic detection methods was included in the groundwater assessment plan. The initial objective was to assess the science of arsenic detection in order to develop a portable, rapid, accurate, and affordable probe; however portable, rapid, accurate arsenic probes were found to be available off the shelf. Most of the tests available are sensitive to 10 ppb, which is the maximum allowable concentration in drinking water set by the EPA.

E. Coli Detection in Water

The Environmental Protection Agency and the State of Virginia mandate the absence of *E. coli* in drinking water (EPA 2005). Thus, an effective probe needs only to determine the presence or absence of *E. coli* bacteria, not its concentration. Yet, the rapid detection of *E. coli* requires complex scientific processes making the development of a probe for field use very challenging. Four criteria were chosen to see if a probe is suitable for wide scale sampling: each test must run in under ten minutes, cost less than fifty cents, be conducted using a portable test kit, and yield accurate, consistent, and reliable results. The PEMC is the only device that fulfills all criteria. It was developed by Dr. Mutharasan of Drexel University and exists only as a prototype. Hopefully, it will be mass produced in the near future. It will probably cost \$10 to \$50.

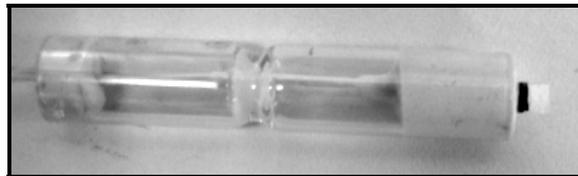


Figure 7. Prototype of the PEMC sensor [Provided by Dr. Mutharasan].

RESULTS AND CONCLUSIONS

The results showed that pH in Ivy was relatively low, with 49% of the wells having a pH below 6.0. However, these low values were expected for this region. The pH distribution is shown below (Figure 8). Nitrate levels on the other hand, were also within the range allowed by EPA.

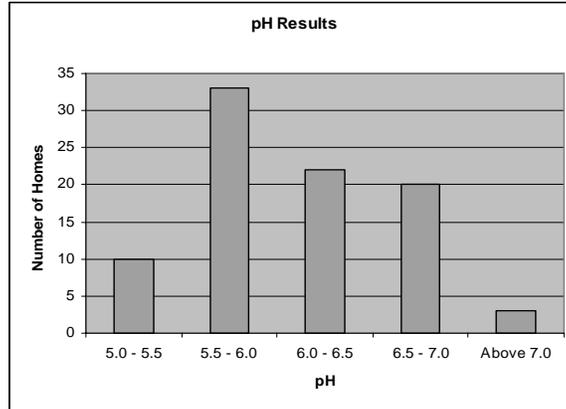


Figure 8. pH Results in Ivy

The testing kit worked well and it could easily be used by a lay person, thereby allowing further field sampling in the future. The GIS mapping system also proved to be an effective way to display the data. The Arsenic and E. Coli probes are promising although not yet ready to be incorporated into the testing kit.

It is equally important that the limitations of the proposed project were discovered. In the Blue Ridge region, groundwater flows are unpredictable. One of the original goals of the project was to assist the Albemarle County Board of Supervisors by predicting precisely how groundwater availability and quality would be affected by additional development in certain areas. It is now known that this is impossible to predict, and that the Board of Supervisors should not rely on specific predictions when issuing building permits.

Future research in this field could address issues encountered as well as study seasonal fluctuations in groundwater quality. The water perception survey results would be tested for some statistically significant difference between the two parties concerned. Studying a larger area or one with a different geologic structure is also recommended. Constant monitoring of a few test wells would also aid the understanding of seasonal trends in groundwater quality and flow rate.

ACKNOWLEDGEMENTS

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ASSESSING THE ECOLOGICAL AND SOCIO-ECONOMIC IMPACTS OF INVASIVE WEED CONTROL ON LAKE CHAPALA, MEXICO

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KEYWORDS: invasive species management, water hyacinth, waterbirds

ABSTRACT

Invasive species are problematic for aquatic systems around the world causing ecological and economic damages. Plans to control non-native species often do not consider the full impacts of management. Despite a strong push toward better understanding invasive species ecology, little effort has been made to investigate the effects of invasive species control on the total ecology of aquatic systems. Lack of research in this area limits our ability to choose the best control strategy and increases the risk of further disturbing ecological balance. This paper discusses a meta-analysis approach for estimating the ecological and socio-economic impacts of various control methods for water hyacinth, a non-native floating macrophyte, in Lake Chapala, Mexico. It also describes the methodology and preliminary results from the first two field sampling periods designed to characterize Lake Chapala's bird community and investigate bird use of water hyacinth in order to predict potential control-induced effects.

INTRODUCTION

Management of aquatic invasive species is a complex process that is further complicated by the intersection between human and ecological needs. A substantial amount of effort has been dedicated to understanding and predicting the impacts of invasive species on non-native ecosystems in order to develop management plans (Parker *et al.* 1999). However, little effort has been given to evaluating the success or failure of different control plans (Klussman *et al.* 1988). Unfortunately, the lack of such research limits our understanding of the full impacts of control. This limitation leaves aquatic systems vulnerable to more ecological disturbance and, in extreme cases, irreparable damages.

In this paper, I discuss the need for a holistic approach to invasive species research. My current research focuses on estimating the potential ecological and socio-economic impacts of water hyacinth control on Lake Chapala, Mexico. Lake Chapala is the largest freshwater lake in Mexico (105,000 ha), yet despite its size and importance to the people of Jalisco and Michoacán state, it has been plagued since the early 1900s with a reoccurring water hyacinth infestation. Water hyacinth has been recognized by many as the worst aquatic invasive plant in the world (Center 1994). It is classified as a floating aquatic macrophyte (Gopal 1987), but in Lake Chapala it is often found along the shoreline. Water hyacinth clusters together as the roots of

individual plants intertwine forming large floating mats that move along the water surface by lake currents and wind. Water hyacinth thrives in eutrophic waters; thus, it is not a surprise that Lake Chapala's nutrient rich waters, due to lack of sufficient water treatment and inadequate land-use practices throughout the watershed, provide an optimal environment for water hyacinth. In general, there are several options for controlling or eradicating aquatic macrophytes like water hyacinth. These include manual, mechanical, chemical and biological removal, as well as prevention plans that reduce the likelihood of invasion. Prior to April 2006, water hyacinth in Lake Chapala was controlled mechanically. April 2006 marked the beginning of a chemical control program that applies glyphosate, a non-selective herbicide, to the water hyacinth (Gonzalez-Orihuela 1996, IMTA 2005).

Water hyacinth is responsible for many water-related problems, including the loss of shoreline access to water, the loss of shoreline habitat, increased navigation difficulty for boaters and fishermen, the clogging of aqueduct and irrigation pipes, and in some extreme cases increased risk water-borne diseases via mosquitoes. However, most of these problems only occur at extremely high densities (Gopal 1987, Navarro and Phiri 2000). On the flip side, water hyacinth provides habitat for waterbirds, shelter and cover for juvenile fish, a sink for heavy metals, nutrients and other contaminants in the water, and has been found to increase densities and species diversity of macroinvertebrates, insects and other fish and bird prey (Masifwa *et al.* 2001, Brendonck *et al.* 2003, Toft *et al.* 2003).

The main goal of my research is to estimate the potential ecological and socio-economic impacts of water hyacinth control on Lake Chapala. In order to do this, I will use a meta-analysis approach to approximate the effects of macrophyte removal on different aquatic ecosystems. The meta-analysis approach will be discussed in greater detail in the methods section. The results of this meta-analysis will be adapted for Lake Chapala by developing a current ecosystem model of the lake based on recent lake research pertaining to water chemistry, plankton dynamics, and community ecology of macroinvertebrates, fish and birds. Little is known about the resident waterbird and migratory bird communities of Lake Chapala; hence, I have developed a field component designed to fill this information gap. The remainder of this paper will briefly discuss the motivation for the meta-analysis component and describe the methodology, preliminary results and implications of the resident bird research mentioned above.

METHODS: META-ANALYSIS

Currently, there is no standard protocol for managing invasive species. Decisions are made on a site-by-site basis reflecting the interests of the major stakeholders. They do not necessarily reflect what is optimal for all interests, and therefore can potentially cause more damage. Furthermore, the success of such management decisions is often not evaluated holistically and the results not made public, which can contribute to more misguided management. Rather than risking further ecological disturbance, I propose a meta-analysis that will examine the results from other lakes with similar problems. The goal of such an analysis is to provide managers with a better picture of the problem before causing further damage through erroneous control efforts. Meta-analysis uses quantitative methods to compare and combine results from analyses that are similar but from separate studies. It provides an estimate for the magnitude of an observed effect which could be more helpful for management decisions than traditional

significance testing (Fernandez-Duque and Valeggia 1994). For example, an effect estimate would allow us to determine which components of an ecosystem may change the most rather than just those that experience a pre-determined change (alpha value).

I plan to use a meta-analysis approach specifically to examine the effects of control on nutrient concentrations, plankton production, abundance and species diversity of macroinvertebrates and fish. Although results from this analysis will not guarantee the same responses within Lake Chapala, it will help determine how common these impacts may be across the board. The results of the meta-analysis will be used to determine which aspects of Lake Chapala's ecosystem are most likely to be affected by different proposed control options.

The ability to use meta-analysis is limited to those areas that have been sufficiently studied and published. Unfortunately, the bird community of Lake Chapala is not well documented, nor has sufficient research been conducted on the relationship between water hyacinth and birds. However, we know bird distributions are influenced by prey availability and that aquatic vegetation provides habitat for aquatic invertebrates (Masifwa *et al.* 2001, Toft *et al.* 2003). The following is the methodology developed to approximate the bird community of Lake Chapala, habitat use and how they respond to the presence and absence of water hyacinth.

METHODS: RESIDENT WATERBIRD USE OF WATER HYACINTH

The first objective of this field component is to increase the local knowledge of resident birds on Lake Chapala and their preferred habitat. The second objective is to investigate the effects of water hyacinth on the birds in order to develop a working hypothesis about how different methods of water hyacinth control may affect the bird community.

In June 2006, I visited eleven sites around Lake Chapala four to seven times each. During each visit I identified and counted birds by species using binoculars and a spotting scope. Each observed bird was recorded in association with a habitat type. Six major habitat types were established, including floating water hyacinth clusters, shoreline water hyacinth, cattail, trees and stumps, shoreline mud or short grass, and an "other" category that included different species of tall grass, water and air observations. I randomly selected these sites taking into consideration the ability to access the lake and the dominant vegetation type. Out of the 11 sites visited, six were classified with water hyacinth as the dominant vegetation and five others were non-water hyacinth sites. In September and October 2006 these sites and 13 new sites were surveyed using the same methodology. Figure 1 illustrates the location of sites around the lake. Five of the new

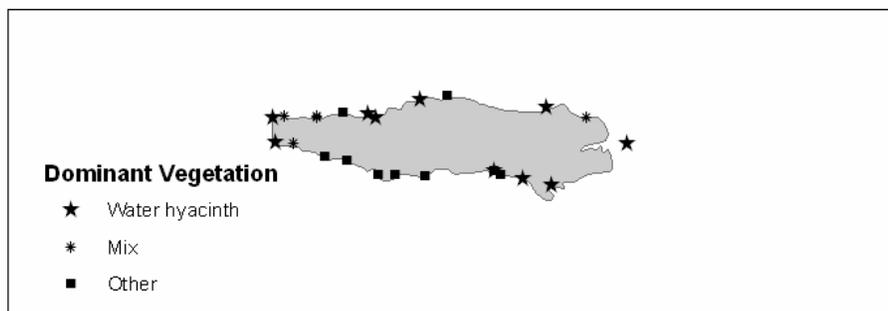


Figure 1. Bird observation sites around Lake Chapala, MX

sites were classified as water hyacinth-dominant, five were non-water hyacinth-dominant, and three others varied during the survey due to moving patches of water hyacinth. Data collected will be analyzed statistically to estimate the relationship between water hyacinth presence and bird abundance (and species diversity). Preliminary results based on two sampling periods are presented in the next section.

RESULTS: RESIDENT WATERBIRD USE OF WATER HYACINTH

More than twenty species of birds were identified at Lake Chapala during June, September and October 2006. Table 1 list these birds and notes * the top five common species observed. Species diversity varied with site ranging from 2.25 to 9.8 species per site-visit. Seven of the eight sites with less than four species observed (average) were classified as non-water hyacinth-dominant. There was also notable difference in mean bird abundance between non-water hyacinth ($\bar{s}_1 = 20.09$ birds) and water hyacinth-dominant sites ($\bar{s}_2 = 40.86$ birds) during both sampling periods. A difference in mean site abundance was also detected between sampling periods. Sampling periods varied in temporal season (dry/rainy) and in treatment. Herbicide had been repeatedly applied for more than a month prior to the first sampling period in June 2006, and the second sampling period began two months after the last spraying.

Table 1. Resident birds observed at Lake Chapala 2006

Great Egret *	Great Blue Heron *	Black-Crowned Night-Heron *	Snowy Egret *
Cattle Egret *	American Coot	Tricolored Heron	Purple Gallinule
Green Heron	Common Moorhen	Tern (Caspian and Least)	Black-Necked Stilt
Little Blue Heron	Least Glebe	Amazon Kingfisher	Mexican Duck
Killdeer	Northern Jacana	Double-Crested Cormorant	White-Faced Ibis

Overall, there was more water hyacinth in the lake during the second sampling period than during the first. This is likely due to the increased precipitation and inflow of nutrients into the lake from the Lerma River and surrounding shoreline towns. Satellite imagery from 2005 suggests a similar seasonal increase in water hyacinth coverage that was not related to a chemical control program (CEAS 2005, 2006). All but one of the original eleven sites kept dominant vegetation classes across seasons, and three of the new sites demonstrated within-sampling period variability with respect to dominant vegetation.

DISCUSSION

Understanding the full impacts of any type of ecosystem management is critical to making wise management decisions. For Lake Chapala, the lack of clear and reliable data limits the ability to determine best control practices. Success of a given method of control is heavily based on how we perceive success and failure. A common estimator of successful management is cost effectiveness (*i.e.*, how many acres of water hyacinth can be removed for given cost); however, this method often fails to consider the full time-scale for treatments. For example, mechanical control requires more replications in a given season than herbicides. It also does not take into account the full effects of the control method. The purpose of the research reported here is to move toward a more holistic evaluation of invasive species control by considering ecological and

socio-economic impacts of control. By doing this we should gain a clearer understanding and estimation of a management plan that optimizes desired returns.

We see in the preliminary results presented above that resident waterbirds do indeed use water hyacinth. It was found that, on average, more birds were associated with water-hyacinth dominant sites than other sites. However, overall there were fewer birds observed in September-October sampling period, near the end of the rainy season when water hyacinth densities were greater along the shoreline. It is uncertain whether this difference is attributed to natural seasonal differences or the effects of chemical control during June. Other helpful information would be the change in available water hyacinth habitat between seasons. Satellite imagery and aerial photos are currently being used to assess changes in water hyacinth availability at the site-level in order to increase the power to detect treatment effects. This determination is further complicated by the temporal variability associated with floating water hyacinth mats. During the September–October sampling period major changes to water hyacinth availability was common; some sites experienced rapid changes occurring during a twenty-minute observation period.

The next questions for this research are to what extent birds use water hyacinth when other options are available? Do they prefer water hyacinth, occupy it equally, or less than other available vegetation types? Do certain species appear to occupy water hyacinth more than others? Answers to these questions will provide a better approximation for how water hyacinth control efforts may impact the resident and migratory bird community specifically.

As mentioned earlier, Lake Chapala is the largest freshwater lake in Mexico and as such we may expect it to support a large population of waterbirds and wintering migratory birds. Although water hyacinth is a non-native plant species, it has been established in the lake for nearly a century. If we were to consider this in a management decision, and accept the likelihood that water hyacinth will continue to exist in the lake as long as nutrient levels are high, then the decision to manage water hyacinth in the lake may take on a new meaning. For one, it requires the acceptance that this invasive will not likely be eradicated from the system before nutrient levels are reduced. Second, the lake may be benefiting ecologically and economically from water hyacinth if we take into account the enriched bird life that could enhance ecotourism and increased available nursery habitat for fish in the lake.

Although deciding how to control water hyacinth will inevitably be subjective, understanding how the lake responds to the presence and subsequent control (absence) of the weed can increase the science behind the management. At the end of this research project, I hope to provide a holistic risk assessment for water hyacinth control and, in the process, develop a framework for estimating the impacts of other invasive species management plans.

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**ENVIRONMENTAL METABOLOMICS:
A NEW APPLICATION FOR WATER QUALITY MONITORING**

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The impact of cadmium (Cd), 2,4-dinitrophenol (DNP) and N-ethyl-maleimide (NEM) shock loads on the composition of the soluble fraction of activated sludge microbial communities was analyzed by gross biomolecular analyses and metabolic footprinting (liquid chromatography-mass spectrometry (LC-MS) techniques). Fresh mixed liquor was divided in four different batches and subjected to no addition of chemical (control) and spike additions of Cd, DNP or NEM. The results indicate that there was a significant release of biomolecules (proteins, carbohydrates and humic acids) from the floc structure into the bulk liquid due to chemical stress. Using discriminant function analysis (DFA) with genetic algorithm variable selection (GA-DFA), the samples subjected to the different conditions were able to be differentiated, thereby indicating that the footprints of the soluble phase generated by LC-MS were different for the four conditions tested and, therefore, toxin-specific. These footprints, thus, contain information about specific biomolecular differences between the samples, and we found that only a limited number of m/z (mass to charge) ratios from the mass spectra data were needed to differentiate between the control and each chemical toxin-derived samples. The discriminant m/z ratios were also toxin-specific. Since the experiments were conducted with mixed liquor from four distinct wastewater treatment plants, the discriminant m/z ratios may potentially be used as universal stress biomarkers in activated sludge systems. This research may serve as a model for similar studies with other environmentally-relevant complex microbial communities that may lead to the development of new monitoring tools for engineered and natural water environments.

REAL-TIME PCR DETECTION OF *LEGIONELLA PNEUMOPHILA* IN WATER

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KEY WORDS: *Legionella pneumophila*, real-time PCR, water, contamination

ABSTRACT

Legionella pneumophila is a ubiquitous aquatic bacterium. Aerosols generated from contaminated water can cause severe pneumonia in immunocompromised individuals. Rapid, sensitive and quantitative detection is needed for the prevention of infection. Using real-time PCR assay and targeting the *L. pneumophila*-specific *mip* gene, we could specifically detected as few as 1.3 copies of a *mip* gene in a 50- μ l reaction from serially diluted *L. pneumophila* genomic DNA. However, this high sensitivity brought false positive problem due to the commercial reagent water that harbors low level contamination by *L. pneumophila* DNA. This problem can be overcome by either treating the reagent water or subtract the contamination from PCR reagents from the sample (albeit with diminished sensitivity). In conclusion, real-time PCR remains a robust method for rapid, sensitive and quantitative *L. pneumophila* assay.

INTRODUCTION

L. pneumophila is commonly found in the aqueous environments and replicates in freshwater amoeba and cilia. Aerosols generated from contaminated water can cause pneumonia in people with weakened immunity. Infections have been related to air conditioning, grocery vegetable mist, whirlpool spa, or industrial cooling towers. Large outbreaks get media attention, but the majority cases are sporadic community infections that are undiagnosed. Early detection of the bacterium is important in the prevention of *L. pneumophila* infection. The classic culture method requires 4-7 days and cannot detect Viable But Non-Culturable (VBNC) bacterium that can be orders magnitude higher than the viable, and remain infectious. Polymerase Chain Reaction (PCR) which targets a genetic marker, dramatically improves the speed, sensitivity and detects the VBNC. More recently, Real-time PCR, which gives quantitative results, is most useful for early detection and risk assessment. Here, we report the application of real-time PCR detection of *L. pneumophila*, we found the assay is specific, highly sensitive, but can be subject to artifacts. However, as shown in our work, this problem can be overcome.

METHODS

Two types of commercial reagent waters were used: 1. *Sigma water* (BioChemika, for molecular biology, Sigma), treatments: deionized from Jersey City water supply, diethylpyrocarbonate (DEPC) was added to inactivate RNase, and autoclaved; 2. *Fisher water* (W5-1, HPLC grade, Cat. No.018709, Fisher), treatment: deionized, tested for UV and visible light absorption, and residue evaporation to be no more than 1 ppm.

L. pneumophila (ATCC 33152) genomic DNA was extracted from Buffered Charcoal Yeast Extract culture using Qiagen kit (Genomic-Tip 20/G, Valencia, CA). DNA concentration and purity were determined by absorbance at 260 and 280 nm using a UV spectrometer (Beckman-Coulter CyberVision C50, Fullerton, CA) to be 0.18 mg/ml, and 1.8, respectively. DNA concentration was used to calculate genome copy number based on the *L. pneumophila* genome size, 3,397,754 bp (Chien *et al.* 2004).

The primers we used were modified based on those described by Ballard *et al.* (2000) to increase PCR stringency. They were forward primer: 5'-AACCGATGCCACATCATTA-3' and reverse primer 5'-CTTGCATGCCTTTAGCCA-3', designed to amplify a 128-bp fragment of the *mip* gene, specific to 15 of the 16 known serotypes of *L. pneumophila*. A BLAST search of the GenBank database demonstrated a high predicted specificity, with the only cross-reacting bacteria being *L. worsleiensis* (GenBank accession number LWU60164) and *L. fairfieldensis* (GenBank accession number LFU60164). The primer pair was synthesized by IDT (Coralville, IA).

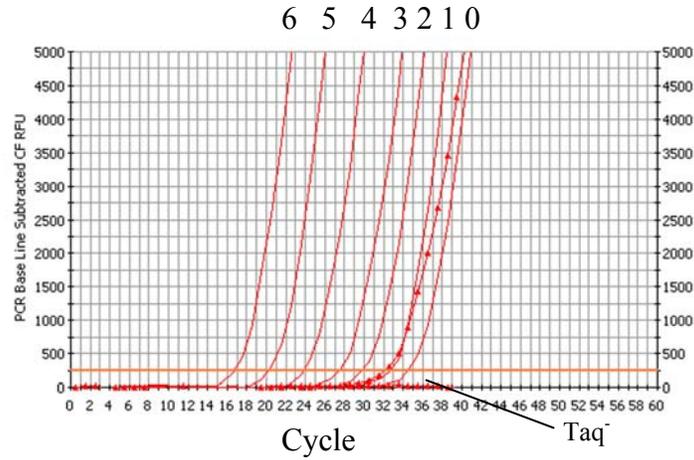
The amplification reaction volume was 50 μ l, which consisted of 5 μ l 10 X PCR buffer, 3.6 μ l of 25 mM MgCl₂, 0.75 μ l of each 20 μ M primer in *Sigma water*, 3.2 μ l of 12.5 mM dNTP in *Sigma water*, 5 μ l of 1: 50,000 diluted SYBRTM Green (Molecular Probes, Eugene, OR), 0.25 μ l of 5-U/ μ l *ampliTaq*^{Gold} (Perkin-Elmer), 5 μ l serially ten-fold diluted *L. pneumophila* genomic DNA, and 26.45 μ l H₂O. Negative controls with no template DNA (replaced with water) were performed for each reaction series. Except specified, the water used in PCR reactions were *Sigma water*, or *Fisher water*.

Amplification and detection were performed using iCycler iQ (real-time PCR detection system, Bio-Rad, Hercules, CA). Real-time PCR consisted of ten minutes at 95°C for *ampliTaq*^{Gold} activation, 50 cycles of two-step PCR amplification (15 s at 95°C and 1 minute at 63.5°C) and a final cooling step at 4°C for 5 minutes. Post-PCR data were analyzed by the iCycler iQ optical system software Version 3.0. 1 μ l of real-time PCR reaction was analyzed by gel electrophoresis, the gel was stained in 1:10,000 diluted SYBR Gold for 5 minutes to visualize the DNA.

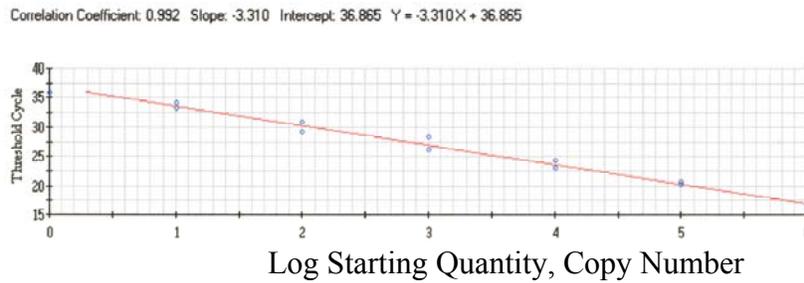
RESULTS

Real-time PCR using *Sigma water* could detect as few as 1.3 copies of *L. pneumophila* genome in 50 μ l of reaction (A, B, Figure 1). However, the negative control which contained no template DNA, also showed a cycle of threshold (C_T) corresponding to about thirteen copies of genome (A, Figure 1), and DNA product of the appropriate size was detected by gel electrophoresis (C,

A.



B.



C.

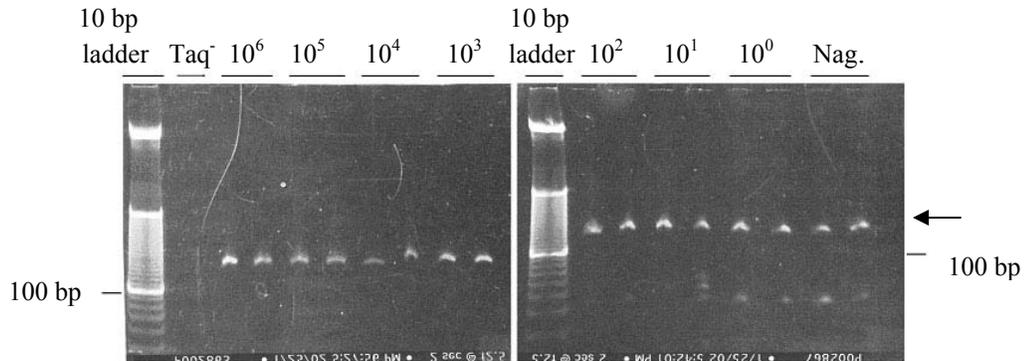


Figure 1. Real-time PCR Standard

A. Amplification profiles of ten-fold dilution series of *L. pneumophila* genomic DNA. 6 to 0: 1.3×10^6 to 1.3×10^0 genome per reaction (50 μ l reaction volume); \blacktriangle : negative control reaction in which no *L. pneumophila* genomic DNA was added. Taq^- , negative control in which no Taq polymerase was added.

B. Generation of standard curve from the data plotted in panel A.

C. Real-time PCR products were detected by gel electrophoresis. 1 μ l of 50 μ l real-time PCR product was loaded onto 10% PAGE. Gel was stained in 10,000 fold diluted SYBR[®] Gold for 5 min. Samples: 10^6 to 10^0 , duplicate real-time PCR reactions of 1.3×10^6 to 1.3×10^0 *L. pneumophila* genomic DNA; Nag., negative control reaction in which no *L. pneumophila* genomic DNA was added; Taq^- , negative control in which no ampli Taq^{Gold} was added. 10 bp ladder: molecular weight marker. Arrow indicates position of *mip* gene amplicons.

Figure 1). In separate experiments, similar C_T was produced even when genomic DNA was diluted to 10^{-3} copies, indicating contamination from the PCR reagents used. Contamination persisted when fresh batches of ampliTaq^{Gold} kit, *Sigma water* and *Fisher water* were tested (Shen *et al.* 2006), all using the same method. These observations suggested that persistent minor contamination in PCR reagents.

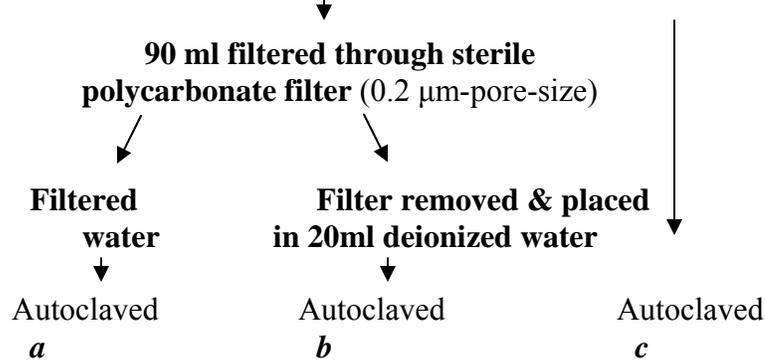
Since *L. pneumophila* is prevalent in natural waters, the bacterium or its DNA could be in the source water from which the commercial reagents are produced. The bacterium can be killed in the purification process or by autoclaving, but DNA may remain in the product. For example, the purified water that we purchased was the products of the manufacturers' own in-house treatment of their local municipal water. The *Sigma water* was deionized municipal water (Jersey City, NJ) treated by DEPC to inactivate RNase, followed by autoclaving (personal communication, Sigma). This treatment does not necessarily remove DNA that might be present in the source water, including *L. pneumophila* DNA. To test this hypothesis, we tried to remove DNA using DNase, and in a separate treatment by Qiagen column adsorption, in both cases without success (data not shown). Then we tried a strategy to remove bacterial cells by filtration (0.2 μm -pore-size polycarbonate filter) prior to autoclaving using deionized water from Socorro municipal supply (A, Figure 2). Real-time PCR using the *Filtered water* (deionized, filtered, and autoclaved) didn't yield a C_T nor was DNA product detected by gel electrophoresis (a, Figure 2). In contrast, *Milli-Q water* (deionized, unfiltered, and autoclaved) yielded a C_T , indicating that filtration before autoclaving removed bacteria and thus reduced the level of *L. pneumophila* contamination below the real-time PCR detection limit, and DNA product was detected by gel electrophoresis in one of the duplicate reactions (c, Figure 2). The *Milli-Q water exposed to a used filter* contained 5.5-fold (from 110 ml to 20 ml) concentrated cells compared to the unfiltered *Milli-Q water*; DNA released in the autoclaving process thus gave an 9-cycles earlier C_T and DNA product was also detected by gel electrophoresis (b, Figure 2). Although, DNA could also be contributed by the Milli-Q (deionization) system and the polycarbonate membrane filter used, the fact that *Filtered water* (deionized, filtered, and autoclaved) didn't yield a C_T in real-time PCR supported our hypothesis that the low level DNA contamination was due to the commercial water (Sigma) used in real-time PCR.

The real-time PCR product in negative control was subcloned and its nucleotide sequence was determined to be identical to the GenBank *L. pneumophila mip* sequence.

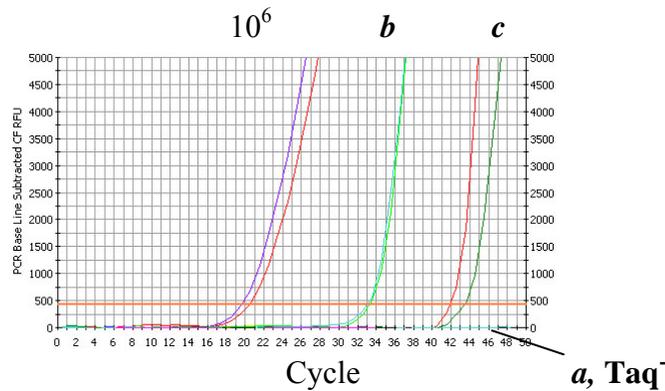
DISCUSSION

Our method can detect 1.3 copies of genome in a 50- μl real-time PCR reaction, similar to the 2.5 CFU/reaction or 10 genomes/reaction reported by other researches (Ballard *et al.* 2000, Wellinghausen *et al.* 2001, Herpers *et al.* 2003), all targeting the *mip* gene. The exquisite sensitivity of DNA-based detection methods presents a potential challenge if the target organism or its DNA is a common contaminant of necessary reagents. It is highly likely that *L. pneumophila* is an even more common contaminant in city water supplies than previously recognized, and this low-level contamination may have gone unnoticed until now due to the lower sensitivity of culture methods. Other researchers encountered similar problem. For example, Van der Zee *et al.* (2002) reported false positives in *Legionella* detection from clinical specimens using PCR method. The authors found that a Qiagen column, a commercial DNA-

A. *Deionized water* by reverse osmosis and ion exchange (Milli-Q, Millipore, Billerica, MA) in Biology Department, NMT, Socorro, NM.



B.



C.

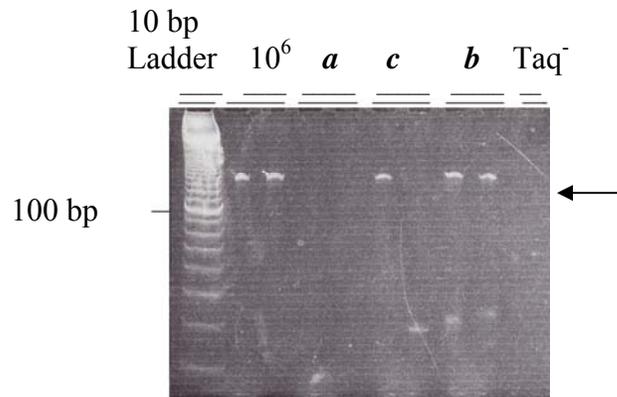


Figure 2. Real-time PCR Negative Control.

A. Three types of water treatment. Deionized water in the biology department (Socorro municipal water) was used to develop the following: *a. Filtered water*: 90 ml deionized water was filtered prior to autoclaving; *b. Milli-Q water exposed to a used filter* used in treatment *a* was aseptically removed and placed into 20 ml deionized water, and autoclaved; *c. Milli-Q water* deionized, unfiltered water was autoclaved.

B. Real-time PCR profile (duplicate reaction). 10^6 : Positive control in which 1.3×10^6 copies *L. pneumophila* genomic DNA were added; Taq^- : negative control, no *ampliTaq^{Gold}* was added; three types of water were tested in negative control (*a*, *b* or *c*) in which 26.45 μ l of the 50 μ l reaction was water received treatment *a*, *b* or *c* as stated in panel **A**.

C. Real-time PCR products examined by gel electrophoresis. 1 μ l of 50 μ l real-time PCR product was loaded onto 10% PAGE. Gel was stained in 1: 10,000 diluted SYBR[®] Gold for 5 min. 10 bp ladder: molecular weight marker. Samples were the same as in panel **B**. Arrow indicates *mip* gene amplicons.

preparation kit, was contaminated with *Legionella* sp. DNA, causing contamination of the genomic DNA preparation. In their case, the columns, flushed with water continuously during the production process may have concentrated DNA on the column. This problem renders PCR as not suitable for the intended *L. pneumophila* detection from clinic samples.

Although low-level DNA contamination in municipal water, commercial water, or reagent kits can be common, and caution should be exercised to be aware of the false positive results when sensitive DNA-based detection methods are used. As we have shown, the water used in DNA extraction and PCR reagents can be purified to remove contaminant DNA. Moreover, real-time PCR is a robust method that can still yield quantitative data (albeit with a diminished sensitivity) even if low-level contamination is not removed from the reagents, because the contamination from PCR reagents can be subtracted from the sample.

ACKNOWLEDGEMENTS

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FTABLE GENERATION METHOD EFFECTS ON INSTREAM FECAL BACTERIA CONCENTRATIONS SIMULATED WITH HSPF

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KEY TERMS: simulation, watershed management, nonpoint source pollution, time series analysis, HSPF

ABSTRACT

The Hydrological Simulation Program-FORTRAN (HSPF) represents discharge using function tables (FTABLES) that relate stream stage, surface area, volume, and discharge. In this study, five FTABLE scenarios were compared to assess their effect on various outputs predicted using HSPF. Four “field-based” FTABLE scenarios were developed using detailed cross-sectional surveys. A fifth “digital-based” scenario was developed using digital elevation data and Natural Resource Conservation Service Regional Hydraulic Geometry Curves. Pair-wise Student’s t-tests were used to compare long-term daily average bacteria concentration, instream bacteria die-off, and violation rates of the Virginia single-sample water quality criterion using the five FTABLE scenarios. The digital-based FTABLE scenarios produced significantly higher instream fecal bacteria concentrations, significantly lower instream bacteria die-off, and significantly higher water quality violation rates. These differences are function of FTABLE generation method and are an artifact of differences in the digital- and field-based volume-discharge relationships that HSPF uses to compute hourly discharge rates.

INTRODUCTION

The Clean Water Act classifies water bodies that do not meet water quality standards as "impaired," and requires TMDLs to bring impaired waters into compliance with water quality standards. According to the U.S. Environmental Protection Agency (EPA), over 40% of the assessed waters in the U.S. are impaired, primarily because of NPS pollution (U.S. EPA 2004). The Hydrological Simulation Program-FORTRAN (HSPF) model is frequently used to develop bacteria impairment TMDLs in Virginia. Simulation with HSPF requires the further delineation of watersheds into subwatersheds and reaches (stream segments) (Boll *et al.* 2003). Each of these reaches requires the input of a Hydraulic Function Table (FTABLE).

When developing FTABLEs for use in HSPF, the modeler is presented with two basic options: use some manner of field data, such as cross-sectional profile surveys, or digital data, such as digital elevation models combined with Natural Resource Conservation Service Regional Hydraulic Geometry Curves. Staley *et al.* (2006) compared simulated daily flow discharge rates

using "field-based" FTABLES (those generated using detailed cross-sectional surveys) and "digital-based" FTABLES and concluded that FTABLE generation method did not significantly affect simulated average daily discharge. The objective of the study reported here is to extend the work of Staley *et al.* (2006) by comparing the effects of the same five FTABLE scenarios (four field-based and one digital-based) used in Staley *et al.* (2006) on simulated instream fecal bacteria concentrations. This study tested the null hypotheses that the simulated daily average instream fecal bacteria concentration does not differ between the five FTABLE scenarios.

METHODOLOGY

The Pigg River watershed drains approximately 906 km² (350 mi²) of Franklin, Henry, and Pittsylvania Counties in south-central Virginia. Land use in the watershed is predominantly forest (72%), followed by agriculture (26%), and residential (2%). For modeling purposes, the watershed was divided into 14 subwatersheds and only one main stream reach was considered in each subwatershed.

Five FTABLE scenarios were analyzed to test the null hypotheses that the method or degree of refinement used to generate HSPF FTABLEs does not affect the model's performance when predicting instream fecal bacteria concentration. The four field-based scenarios used from one to four surveyed stream channel cross-sections averaged together in various combinations to produce an FTABLE for each subwatershed stream reach. Four cross-section profiles (Outlet, 1/3, 1/2, and 2/3) were surveyed in each subwatershed (Figure 1), resulting in 56 cross-section profiles for the entire Pigg River watershed. The combinations of cross-section profiles used to develop the four field-based scenarios are shown in Table 1. The field-based scenarios were designed to illustrate what effect, if any, more or less field data used in FTABLE generation had on simulated instream fecal bacteria concentration. A fifth digital-based FTABLE scenario combined the Natural Resource Conservation Service Regional Hydraulic Geometry Curves for the Piedmont Upland region (NRCS 2004) were combined with U.S. Geological Survey 30-m DEM data (USGS 2004). This digital-based FTABLE scenario uses FTABLEs created using a comparatively simple method to generate FTABLEs. The FTABLEs used in this study were originally generated by Staley *et al.* (2006).

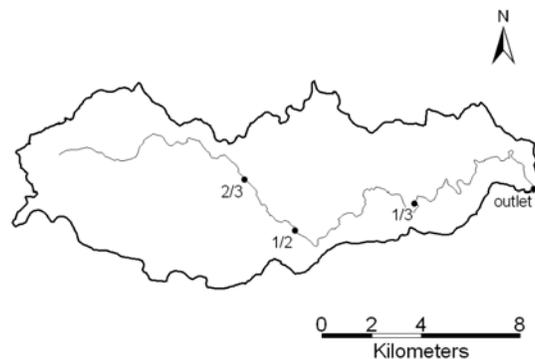


Figure 1. Example cross-sectional profile locations for field-based FTABLE scenarios.

Table 1. The cross-sectional profiles used in processing each field scenario (Staley *et al.* 2006).

Field Scenario	Cross-sectional profiles used
“outlet”	outlet
“half”	outlet, 1/2
“third”	outlet, 1/3, 2/3
“all”	outlet, 1/3, 1/2, 2/3

HSPF was used to simulate instream bacteria concentration, instream bacteria die-off, and discharge at an hourly time step. The final hydrologic parameters used by Staley *et al.* (2006) were also used in this study. Fecal bacteria modeling input parameters used in this study were based on data used in the Pigg River TMDL study (Benham *et al.* 2006). The hydrology and bacteria input parameters did not vary with FTABLE scenario. Simulations to assess each FTABLE scenario were performed using a single HSPF User Control Input (UCI) file, one file per FTABLE scenario.

To create the data needed to compare the effects of the different FTABLE generation methods, Monte Carlo simulations for HSPF were conducted using stochastically generated climate inputs. The CLIGEN program (USDA 1989) was used to stochastically generate 75 synthetic metrological input time series data sets. The period of the metrological time series was 21 years. CLIGEN generates climate data at a daily time step. The time step that was used in HSPF simulations was 1 hour. Therefore, the CLIGEN data needed to be disaggregated from a daily to an hourly time step. The disaggregation procedures used in WDMUtil (Hummel *et al.* 2001) were employed using software developed by one of the authors to convert the daily CLIGEN data to an hourly time step. Using the statistically similar disaggregated climate time series, 75 runs of HSPF were performed for each FTABLE generation method in the Monte Carlo simulations to create the data sets used in the statistical analysis of the simulated response variables. Of the 21 years of response variable output, only the last 16 years were used in the analysis reported here.

HSPF outputs of interest included daily average flow rate or discharge, daily average instream fecal bacteria concentration, and the amount of bacteria die-off that occurred within the watershed stream reaches. Discharge rate and instream bacteria concentration outputs at the Pigg River watershed outlet were averaged over the 16-year analysis period to calculate a long-term daily-average discharge and instream bacteria concentrations for each FTABLE scenario and climatic data set model run combination. Daily average instream bacteria concentrations were used to calculate water quality criteria violation rates, the percentage of time the daily average instream concentration exceeded the applicable water quality criteria (400 colony forming units ‘CFU’ per 100mL). As applied here, HSPF modeled fecal bacteria as a dissolved water quality constituent and used a first order decay to simulate instream bacteria die-off. The hourly instream bacteria die-off amounts were summed to obtain a daily bacteria die-off amount. The daily die-off was summed over the 16-year analysis period obtain the total amount of bacteria die-off that occurred within the stream reach for each FTABLE scenario and climate data set model run combination.

HSPF Monte Carlo simulations of the 5 FTABLE scenarios using the 75 synthetic climate time series data sets resulted in 375 independent observations of long-term daily-average discharge, instream fecal bacteria concentration, bacteria die-off, and violation rate. These response variable data sets fit a Gaussian distribution, thus statistical comparisons of the 5 FTABLE scenarios were made using a pair-wise Student's t-test.

RESULTS AND DISCUSSION

An initial visual comparison of the effect of FTABLE generation method on long-term daily average instream bacteria concentration was made by plotting the 16-years of daily average instream bacteria concentrations simulated using the various FTABLE scenarios against each other to assess if any obvious trend existed. The statistical analysis of the long-term daily average instream bacteria concentrations indicated that the digital-based FTABLE scenario resulted in statistically significant higher ($\alpha = 0.05$) simulated instream fecal bacteria concentrations at the watershed outlet when compared to any of the field-based scenarios (Table 2). The analysis also indicated that there were statistical differences between some of the field-based scenarios. Figure 2 shows box and whisker plots comparing the long-term daily average instream fecal bacteria concentration simulated using the five FTABLE scenarios. It is clear that the digital-based FTABLEs simulated higher long-term daily average instream bacteria concentrations when compared to any of the field-based scenarios. Figure 2 also illustrates that while there may have been statistically significant differences between some of the field-based scenarios, from a practical, modeling perspective the field-based scenarios are essentially equivalent.

Table 2. Pairwise t-test p-values for the comparison of long term average bacteria concentrations simulated by HSPF for each scenario ($\alpha = 0.05$ with a Bonferroni correction).

	all	third	half	out
digital	$< 2 \times 10^{-16}$			
all		1	0.006	0.022
third			0.018	0.058
half				1

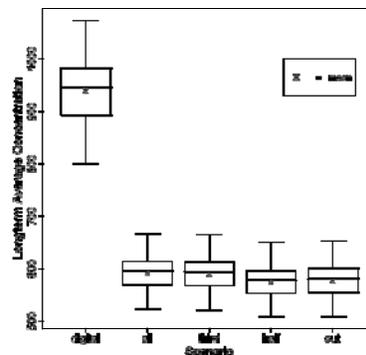


Figure 2. Box and whisker plots summarizing simulated long-term average concentration HSPF output.

We have presented evidence that the differences between the digital- and field-based scenarios are both statistical and practical. However, we have suggested that the statistical differences between the field-based scenarios do not have practical significance. This point is reinforced by analyzing water quality criteria violation rates resulting from each FTABLE scenario. In Virginia, a TMDL pollutant allocation scenario is acceptable only if the state’s “single sample” water quality criteria (400 CFU/100ml) is not exceeded during the TMDL analysis simulation period. Thus, if the field-based FTABLE scenarios are not statistically different in terms of predicted water quality criteria violation rates, any differences in simulated instream bacteria concentrations will have no practical implication. An analysis of the water quality criteria violation rates indicated that while the digital-based FTABLE scenarios do predict significantly higher water quality violation rates than the field-based scenarios ($\alpha = 0.05$), there is not statistical difference among the field-based scenarios (Table 3).

Table 3. Pairwise t-test p-values for the comparison of violation rates simulated by HSPF for each scenario ($\alpha = 0.05$ with a Bonferroni correction).

	all	third	half	out
digital	$< 2 \times 10^{-16}$			
all		1	0.076	0.122
third			0.194	0.294
half				1

Previous research examining the effects of FTABLE generation method (Staley *et al.* 2006) concluded that average daily discharge was not different when comparing digital- and field-based FTABLE generation methods. The results presented here, that digitally generated FTABLEs produce higher instream fecal bacteria concentrations than the field-based FTABLEs may, at first, seem, to contradict the findings of Staley *et al.* (2006). However, upon further analysis the results presented here are not inconsistent.

Instream bacteria fecal concentration at the watershed outlet is a function of bacteria loading to the land surface, bacteria loading directly to the stream surface (direct deposit), and the volume and timing of flow. Bacteria loading and climate inputs were identical for each climate data set and FTABLE scenario combination simulated here. A pair-wise Student’s t-test ($\alpha = 0.05$) was performed to confirm that long-term daily average discharge rate (defined as a 16-year mean of the daily average discharge rates) is not a function of FTABLE generation method. The results of this analysis reaffirmed the conclusions of Staley *et al.* (2006).

To understand the differences found in long-term daily average concentration, the timing of the discharge in the watershed, *i.e.*, the shape of the hydrograph and the implications for instream fecal bacteria residence time must be considered. This point can be illustrated using an example storm hydrograph, Figure 3. Note that the total area under both hydrographs in Figure 3 is equal, but the hourly discharge rates are different. The digital-based FTABLE hydrograph peaks higher and earlier than the field-based hydrograph. This difference in flow timing impacts instream bacteria concentration by altering the instream bacteria residence time. The instream bacteria residence time is shorter for the digital-base FTABLE scenario than the field-based FTABLE scenarios.

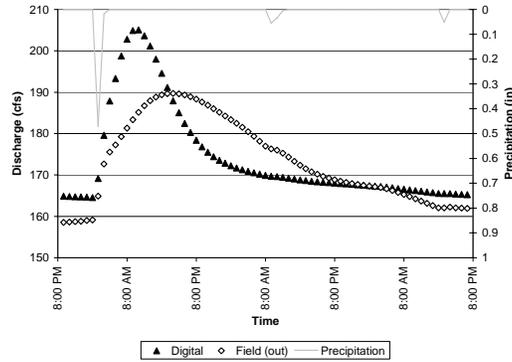


Figure 3. Example hydrograph illustrating differences in digital- and field-based FTABLE scenario simulated hourly discharge rates.

The hypothesis that the digital-based FTABLEs yielded a shorter instream residence time was confirmed by comparing the amount of bacteria die-off that occurred in the Pigg River basin for each of the five FTABLE scenarios. The amount of bacteria that died-off per day was summed for each of the climate date set FTABLE combination simulations. As with the other response variables, a pair-wise Student’s t-test was used to compare the five FTABLE scenarios. As was expected the digital-based scenario (shortest residence time) allowed for significantly less ($\alpha=0.05$) instream die-off when compared to the field-based scenarios (Table 4).

Table 4. Pairwise t-test p-values for the comparison of total bacteria die-off simulated by HSPF for each scenario ($\alpha = 0.05$ with a Bonferroni correction).

	all	third	half	out
digital	$< 2 \times 10^{-16}$			
all		1	0.030	$< 2 \times 10^{-16}$
third			0.420	8×10^{-16}
half				1.5×10^{-10}

SUMMARY AND CONCLUSIONS

The objective of this study was to examine the effects of FTABLE generation method on instream fecal bacteria concentrations simulated using HSPF. FTABLES can be generated using digital-based methods or more time consuming field-based methods (topographical surveys). The FTABLES used in this study were originally generated by Staley *et al.* (2006). HSPF was executed on an hourly timestep for five FTABLE scenarios (one digital-based and four field-based). HSPF outputs included daily average discharge, daily average bacteria concentration, and daily bacteria die-off, which were used to calculate the response variables: long-term average bacteria concentration, water quality criteria violation rates, and total bacteria die-off. Simulations using digitally generated FTABLES yielded significantly higher long-term average instream fecal bacteria concentrations when compared to the simulations performed using the field-based FTABLES. The higher concentration levels also resulted in the digital-based FTABLE simulations yielding significantly higher water quality criteria violation rates. With respect to the field-based scenarios, there was no practical difference with respect to long-term instream bacteria concentration between the four scenarios and no statistical or practical difference in terms of water quality criteria violation rate. This research implies that using

digitally generated FTABLES may make achieving an acceptable TMDL pollutant allocation scenario more difficult. It also illustrates that developing more refined field-based FTABLES *i.e.*, collecting more than one topographic cross-sectional survey has very little effect on predicted instream fecal bacteria concentrations. Additional research is needed to investigate the accuracy of FTABLES generated using the digital- and field-based methods with respect to observed data.

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ASSESSING ALTERNATIVE METHODS USED TO MODEL FECAL COLIFORM DIRECT DEPOSIT

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ABSTRACT

The freshwater bacteria water quality standard in Virginia specifies in-stream concentration limits and includes both an instantaneous or single sample criterion and geometric mean criterion. Direct deposition (DD) of bacteria is often a significant contributor to violations of the freshwater bacteria standard. This effect is exaggerated by limitations in water quality simulation software that cause erroneously high predictions of bacteria concentrations when small amounts of bacteria are deposited directly in the stream during low-flow conditions. The goal of this study is to evaluate the impact of three different 'low flow' DD simulation methods on bacteria source reductions: DD cut-off as a function of livestock behavior, flow stagnation, and stream reach surface area. The three methods will be investigated using watersheds with previously developed bacteria impairment TMDLs, and violation rates will be used as the response variable. A One-Way ANOVA factorial design will be used to compare treatments statistically.

THE REMOVAL OF ESTROGENS IN DIFFERENT DAIRY MANURE TREATMENTS

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ABSTRACT

The effects of three manure treatments on estrogen removal (E2 and E3) were examined in this study. Samples were taken from a full-scale manure handling system incorporating manure separation and aeration, an anaerobic digester receiving dairy manure, and anaerobic slurry from three commercial dairy farms. E2 and E3 were analyzed using enzyme linked immunosorbent assay. Significant differences were found in estrogen concentrations in land-applied manure in the three treatments ($P < 0.01$). The system combining removal of solids and aeration of liquids significantly reduced the total mass of E2 and E3 ($P < 0.05$). The solid separation in this system did not reduce estrogen mass-balance ($P > 0.10$) in liquid effluent, but the sequential aeration substantially contributed to estrogen removal. E2 ($P < 0.05$) but not E3 ($P > 0.05$) was significantly decreased after anaerobic digestion.

INTRODUCTION

In the last few decades, estrogens in the environment have induced great concern and interest for their potent endocrine disrupting effects. Significant estrogens have accumulated in the environment because of increased population and intensity of animal production. At concentrations as low as ng per liter of water, estrogens can cause dangerous consequences to aquatic species (Colborn *et al.* 1993). For example, vitellogenesis and feminization in male fish are attributed somehow to the presence of these estrogenic compounds (Jobling *et al.* 1998).

To meet the strict federal and state laws in terms of nutrients management from farm animal waste, manure treatments are necessary to decrease nutrients loading to the environment. On the other hand, these treatments may affect estrogen loading in the manure by different operations like solid separation, liquid aeration, and anaerobic digestion. It is important to quantify the effects of manure treatments on estrogen contents for further balancing nutrients and estrogen management. But most studies about estrogen degradation have been done in municipal

wastewater treatments. With conventional activation sludge system (aeration) in municipal treatment plants, E2 was reduced from 15.6 ng/L in the influent to 1.8 ng/L in the effluent (Servos *et al.* 2005). When incubated under anaerobic conditions (methanogenic) with lake water and sediment, 99-176 µg/L per day of E2 (5 mg/L originally) was transformed to estrone depending on the availability of electron acceptors (Czajka and Londry 2006).

OBJECTIVES

The objectives of this study were to set up an effective method for estrogen quantification in manure samples and to explore the effects of three types of manure treatments on estrogen removal.

MATERIAL AND METHODS

Sample collection

Flushed dairy manure samples were collected on a full-scale manure handling system receiving manure from about 140 dairy cows at the Virginia Tech dairy center. This system employs a mechanical separator to separate manure liquids from solids, a short retention time anaerobic settling basin to remove further solids, and three aerated tanks in sequence. Total retention time in the system is approximately 180 days. The effluent from the third tank is reused to flush the barn, or land-applied to crops via irrigation. The main slurry from the barn (MS), separator effluent (SP), settling basin effluent (SB), effluent from the first two tanks (T1, T2), and irrigation water (effluent from the third tank, T3) were collected monthly from July to December 2005.

The influent (IN) and effluent (EF) of an anaerobic digester on a commercial dairy farm raising 2,400 cows were collected from August 2005 to January 2006. Anaerobically stored manure slurry samples (M1, M2, and M3) were collected from three other commercial farms during fall land application in 2005. All samples were stored in capped glass jars at -20 °C before analysis.

Sample preparation

For estrogen analysis, 1 ml or 1 g homogenized sample was thoroughly mixed with 1.5 ml chloroform and 1.5 ml NaOH (1 N) and centrifuged for 20 min at 2500 g. 1 ml of the supernatant alkaline phase was removed and neutralized with 180 µl 90% acetic acid. The sample was further extracted with 3 ml toluene twice. The extraction procedure is mainly based on Hoffmann *et al.* (1997). The combined toluene was then evaporated to dryness with a gentle nitrogen gas stream.

Estrogen analysis

The concentrations of 17β-estradiol (E2) and estriol (E3) in the samples were assayed with commercial ELISA kits (Assay Design, MI, USA). These are competitive immunoassays for the quantitative determination of E2 and E3 in environmental samples. The intraassay and interassay coefficients of variation for E2 were 5.2% and 10.2%, respectively, and 6.0% and 7.4% for E3, respectively. The recoveries were 95-110% and 70-75% for E2 and E3, respectively.

Statistical analysis

The effects of the manure handling system and anaerobic digestion on the concentrations of E2 and E3 were analyzed according to the following model:

$$Y_{ijk} = \mu + S_i + L_j + SL_{ij} + e_{ijk},$$

where μ = overall means,

S_i = season effects,

L_j = location effects,

SL_{ij} = interaction between seasons and locations, and

e_{ijk} = residual error.

Data were analyzed using the MIXED procedure of SAS (SAS Institute 1999). Significance was declared as $P < 0.05$.

RESULTS AND DISCUSSION

1. The estrogen concentrations of land-applied samples

The irrigation water (T3), anaerobic digester effluent, and anaerobic slurry will be land-applied directly, and the estrogen concentrations in the three kinds of samples are compared in Figure 1. There are significant differences between the three manure treatments ($P < 0.01$) for both E2 and E3. The lowest E2 (360 pg/ml) and E3 (167 pg/ml) concentration in irrigation water is mainly due to the dilution effect when flushing the dairy barn.

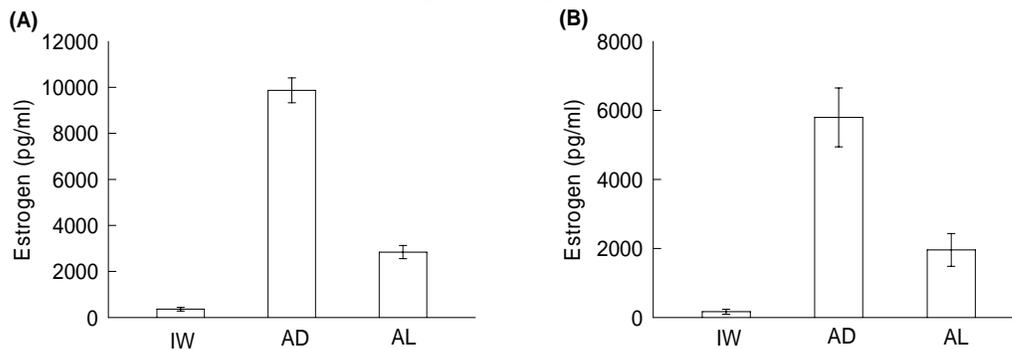


Figure 1. Mean concentrations of E2 (A) and E3 (B) in land-applied samples, e.g., irrigation water (IW, n=12), anaerobic digester slurry (AD, n=9), and anaerobic slurry (AL, n=3). The error bars represent standard errors as in the following figures.

2. The effects of mechanical separation on estrogen removal

The mass balance (concentration * flow) of E2 and E3 in MS (separator influent) and SP (separator effluent) per day is shown in Figure 2. Across all seasons, 86.0% of E2 (53.3 mg vs. 45.7 mg) and 77.2% of E3 (38.4 mg vs. 28.1 mg) in MS are still present in separator effluent, and the effect of mechanical separation on estrogen removal efficiencies was not significant for both E2 and E3 ($P > 0.10$). Only 0.24% of E2 and 0.46% of E3 in the main slurry are transported into the solid mass after separation. There is 14% (E2) and 23% (E3) discrepancy between the sum of estrogens in separator effluent and solid mass and estrogens in separator influent. The aeration process in reception pit holding separator influent, sampling error for fresh solids, and leachate loss from fresh solids are probably responsible for the unaccounted part of estrogens.

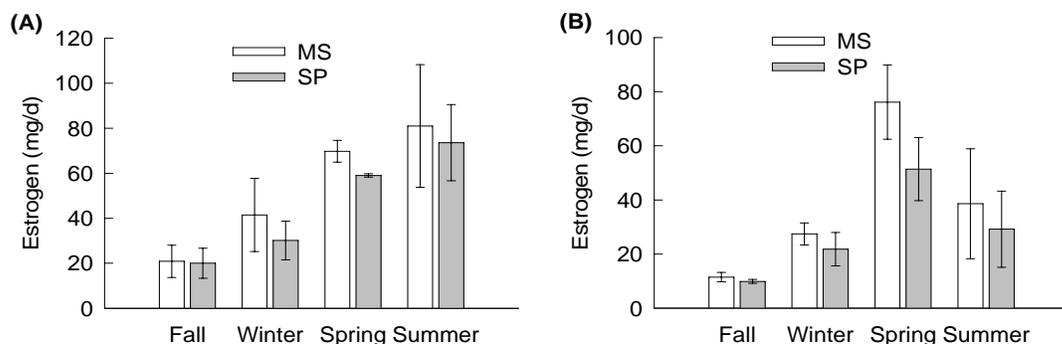


Figure 2. The mass balance of E2 (A) and E3 (B) before and after mechanical separation.

3. The effects of sequential aeration on estrogen removal

The mass balance of settling basin effluent (Tank 1 influent) and irrigation water (tank 3 effluent) are shown in Figure 3. The sequential aeration in the three tanks removed both E2 (38.7 mg vs.16.3 mg, $P < 0.05$) and E3 (24.2 mg vs. 5.5 mg, $P < 0.05$) significantly, which means 57.9% and 77.4% removal efficiency for E2 and E3, respectively. The available oxygen from the aeration process is probably aiding in estrogen degradation in the tanks. This result is in agreement somehow with the none or minimal estrogen receptor gene transcription activities in biosolid that had undergone aerobic digestion (Lorenzen *et al.* 2004).

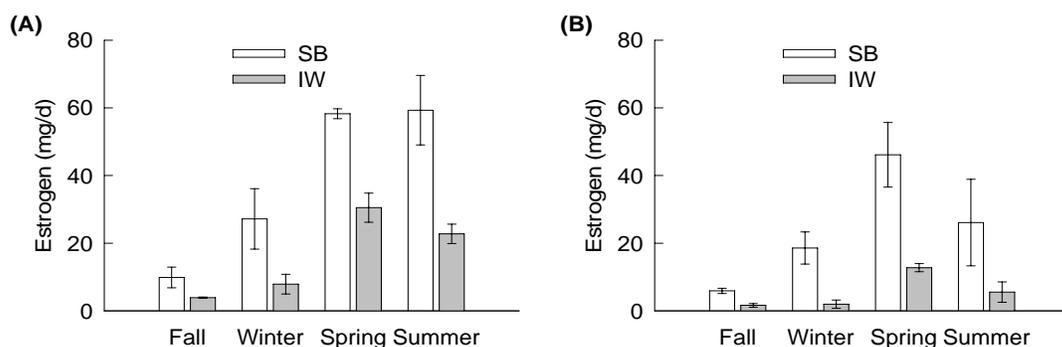


Figure 3. The mass balance of E2 (A) and E3 (B) before and after sequential aeration.

4. The overall effect on estrogen removal in VT dairy manure system

The overall effects of VT dairy manure treatment system on estrogen removal are shown in Figure 4. Both E2 (53.3 mg vs.16.3 mg, $P < 0.05$) and E3 (38.4 mg vs. 5.5 mg, $P < 0.05$) are significantly decreased after the whole system, which means an overall 72.6% and 86.9% of removal efficiency for E2 and E3, respectively. This is in comparison to the efficiencies in normal sewage treatment plants, although the treatment processes and environment conditions may be different. Servos *et al.* (2005) reported 75-98% removal efficiencies of E2 in municipal wastewater treatment plants incorporating the primary and secondary treatment (conventional sludge) together.

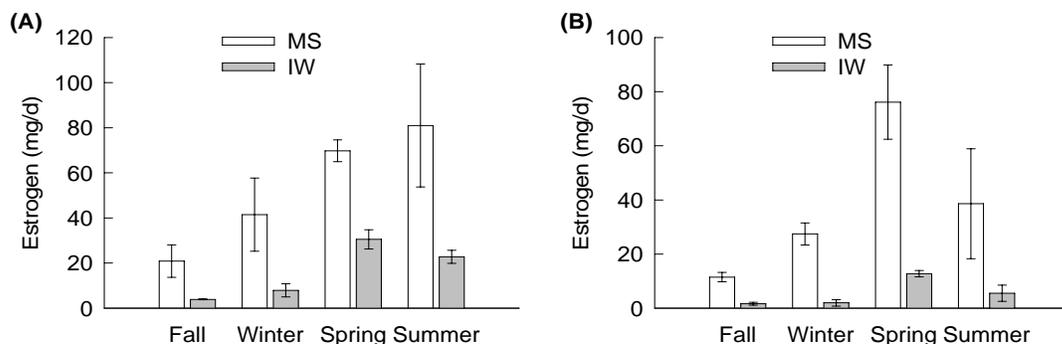


Figure 4. The overall mass balance of E2 (A) and E3 (B) in VT dairy manure system.

5. The effects of anaerobic digester on estrogen removal

The concentrations of E2 and E3 in the influent and effluent of the anaerobic digester are shown in Figure 5. E2 (15.9 ng/ml vs. 9.9 ng/ml, $P < 0.05$) but not E3 (7.9 ng/ml vs. 5.8 ng/ml, $P < 0.10$) is significantly decreased, which means 38.0% and 26.5% removal efficiency for E2 and E3, respectively. The mass balance is not presented here because of the unavailability of flow data. The removal of E2 was observed by Czajka and Londry (2006) in methanogenic conditions, where E2 was readily transformed E1 and probably 17 α -estradiol, and this means reduction in total estrogenic activities. But Lorenzen *et al.* (2004) reported much higher estrogen receptor gene transcription activities in biosolids undergone anaerobic digestion compared to aerobic digestion, which indicates a lower removal efficiency of estrogens in anaerobic digestion.

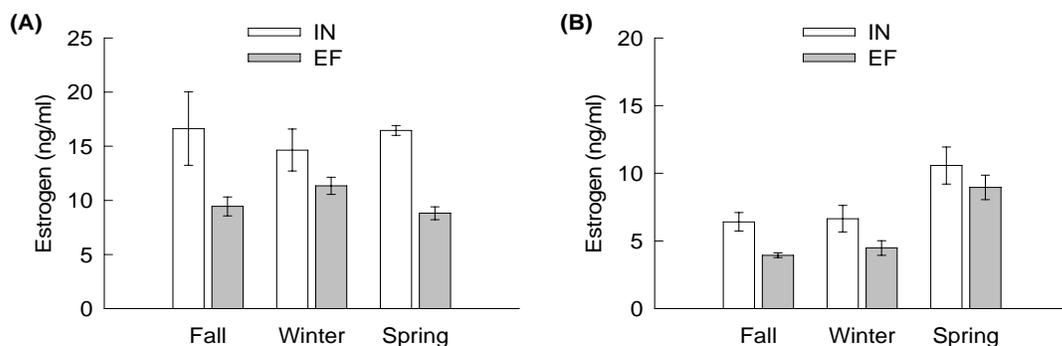


Figure 5. The effect of anaerobic digestion on the removal of E2 (A) and E3 (B) in dairy manure.

CONCLUSIONS

The estrogen concentrations in three different dairy manure treatments were determined with an ELISA method, which is very sensitive, accurate, and proper for the samples within this study. This method does not require tedious sample pretreatment, such as concentration and purification that are generally needed for instrumental analysis. Based on this study, estrogen concentrations in dairy manure samples are substantially decreased with two treatment methods. Further research is needed to clarify the concentrations of estrone and conjugated estrogens that may present in dairy manure.

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WATER QUALITY IN RETENTION PONDS DURING STORM EVENTS

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KEY WORDS: retention ponds, water quality, storms

ABSTRACT

We measured storm-event runoff and determined sediment and nutrient removal efficiency of three retention ponds in James City County, Virginia. We calculated time-specific flows into and out of basins during storm events, and used ISCO automated samplers to collect water for analysis of suspended sediment and nitrogen and phosphorus species. In one pond, the total suspended sediment discharged from the basin during a storm was greater than the amount entering the basin. In the other two ponds, nutrient removal efficiency ranged from 0% to 98% and was variable both by storm and by species. Efficiency of nutrient retention is influenced by watershed characteristics, basin engineering, antecedent conditions, and the intensity and size of storms. Thus, the ability of stormwater retention basins to meet any specified water quality standard cannot be calculated without actual measurement, *i.e.*, engineering solutions do not predict the observed field variation in basin performance.

INTRODUCTION

Housing development disturbs soils and creates impervious surfaces. Runoff from housing developments during storms mobilizes non-point source pollutants including sediment and various forms of nitrogen and phosphorus. Sediments and nutrients are the two most common pollutants in developing watersheds that together cause the greatest impacts on downstream

ecosystem function. Non-point source pollution discharging from developing watersheds is to some extent reduced by the presence of stormwater retention ponds (Mallin *et al.* 2002, Marsalek *et al.* 2002). Although retention ponds are constructed primarily to reduce peak volume discharges of water contributed by runoff from impervious surfaces, the retention of water in basins can facilitate pollutant removal by physical, chemical, and biological processes (Perniel *et al.* 1998). Despite the importance of these secondary benefits to maintenance of water quality in developing watersheds, few standards exist for pollutant removal in stormwater retention ponds.

The current study was completed to determine the pollution removal efficiencies of three stormwater retention ponds in James City County, VA. The ponds were monitored during storm events to calculate the flux of pollutants into and out of ponds. Comparisons of removal efficiencies were made both among ponds and storms. Our goal was to determine whether we could identify common responses by stormwater retention ponds that would assist our understanding of their potential ecosystem service as sites for water quality improvement.

METHODS

Three retention ponds in James City County, VA, were chosen for study. The ponds were less than 10 years old, but were of varying shape and size. All ponds had been constructed to handle runoff from housing developments with 1/8- to 1/2-acre lot sizes. The overflow from each pond discharged to headwater streams in the James River sub-estuary to Chesapeake Bay.

Stormwater monitoring for pond performance required knowledge of water volumes into and out of ponds throughout storm hydrographs. Thus, a continuous record of pond outflow, inflow, and storage was obtained through automated data collection at each retention pond. Pond elevation was measured using a Druck PDCR 1830 pressure transducer installed in a stilling well located in the pond, connected to a Campbell Scientific CR500 datalogger that recorded time and instantaneous pressure every five minutes. The pressure reading was converted to pond elevation through a rating curve obtained through repeated, independent measurement of pond elevation and pressure reading. Elevation was then converted to storage volume and pond outflow using rating curves constructed from data in the as-built engineering plans for each pond. The design elevation/discharge relationship was supplemented with field measurements of pond outflow using the salt dilution method, when possible. From measured pond outflow (Q_{out}) and changes in storage volume ($\delta V/\delta t$), pond inflow was obtained as $Q_{in} = \delta V/\delta t + Q_{out}$.

Water samples for determination of water quality were collected during storms at inflow and outflow locations on each pond. We programmed automated ISCO[®] water samplers to collect water during storms every 15-60 minutes, depending on duration of expected storms. We were unable to use flow-actuation to begin sampling, and had to initiate sampling manually when individual storms began. Samplers collected up to 24 500-ml water samples throughout storm hydrographs. Immediately following storms, the water samples were retrieved and placed on ice prior to analysis.

All water samples were treated identically. First, each sample was passed through a pre-ashed, pre-weighed, 0.45 μm glass fiber filter to remove suspended sediment. The filter was then dried in a 60°C oven and re-weighed for determination of total suspended sediment, then ashed at

450°C for three hours and resuspended in 1N HCl for determination of total particulate phosphorus (Chambers and Fourqurean 1991). The filtrate was analyzed for dissolved inorganic phosphate, nitrate+nitrite, and ammonium using standard methods (Parsons *et al.* 1984).

The time-specific water quality measures were multiplied by the calculated volumes Q_{in} and Q_{out} to obtain total amounts of sediment and nutrient flowing into and out of ponds, which were then plotted over time. Integration over the storm allowed us to calculate and compare the amounts of material carried into the pond and discharged from the pond during individual storm events.

RESULTS

One pond located at Ironbound Village was monitored for a single, short thunderstorm in July 2005. Sediment flux peaked soon after runoff began; throughout almost the entire storm, more sediment flowed out of the pond than was flowing in (Figure 1). For this single storm, we calculated a negative removal efficiency: the pond released some 2800 grams of sediment more than it received from runoff, indicating that resuspension of sediments from the basin was contributing to outflows. Instead of being a sink for sediment, the pond was functioning as a source of sediment (and total particulate phosphorus) and impairing downstream water quality. The fluxes of dissolved nutrient species into and out the Ironbound Village retention pond varied dramatically throughout the storm, and as a result we were unable to calculate net release or removal. Interestingly, however, the concentration of nitrate+nitrite, although low in inflow water, tended to be higher in outflow water by the end of the storm, suggesting that microbial transformation of ammonium to nitrate was occurring, perhaps associated with the resuspension of sediments from the bottom of the pond.

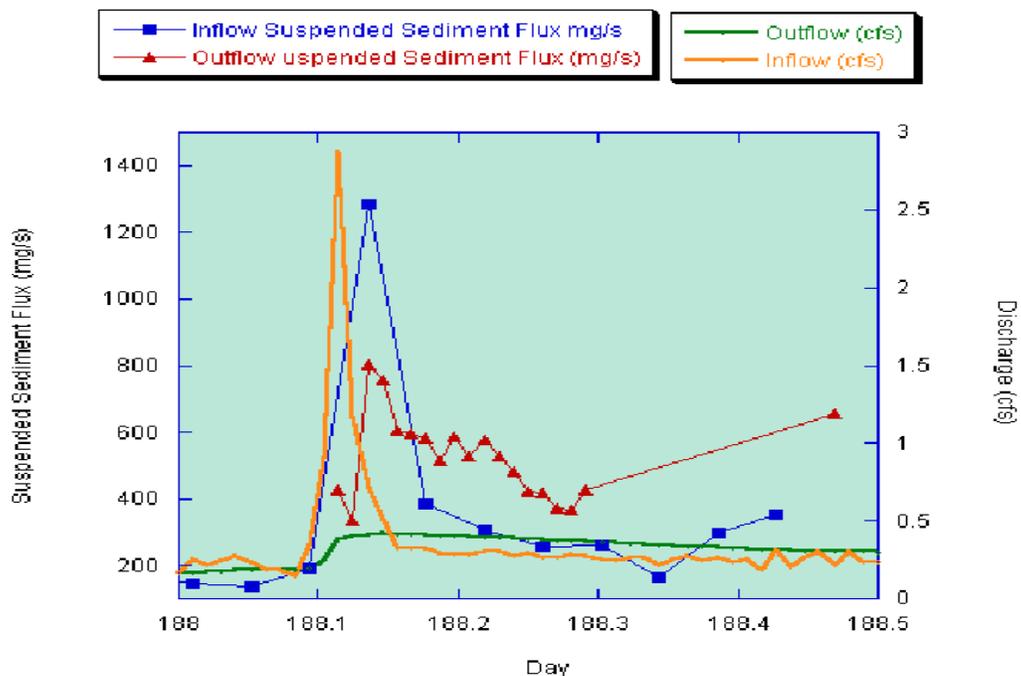


Figure 1. Flux of Suspended Sediment in Ironbound Village Retention Pond During a Storm on 7 July 2005.

The ponds located at Pointe at Jamestown and at Mulberry Place were monitored for two storms in June and July 2006. Calculated removal efficiencies for the first storm are summarized in Table 1 and were generally higher for Mulberry Place than for Pointe at Jamestown. For both ponds, a higher percentage of dissolved phosphorus load was removed relative to the particular phosphorus load. Nitrate+nitrite loads were more than one order of magnitude larger than ammonium loads, and removal efficiencies for both ponds were similar at 62 and 63%. Half the suspended sediment load of over 30 kg was removed by the Pointe at Jamestown pond.

Table 1. Summary of removal efficiencies for two retention ponds during a storm on 27 June 2006.

Location	Rainfall (inches)	Parameter	Total In (g)	Total Out (g)	Removal Efficiency
Mulberry Place	0.38	Particulate P	20	12	38%
		Dissolved P	184	24	87%
		Ammonium	53	12	78%
		Nitrate+Nitrite	865	332	62%
Pointe at Jamestown	0.33	Particulate P	36	33	8%
		Dissolved P	155	84	46%
		Ammonium	75	69	9%
		Nitrate+Nitrite	2,444	869	63%
		Suspended Sed	30,080	15,070	51%

For the second, larger storm in July 2006, these patterns in nutrient loads and removal efficiencies changed dramatically (Table 2). For both ponds, particulate phosphorus removal was greater than for the earlier, smaller storm. Dissolved phosphorus removal efficiency was again fairly high, but ammonium removal was poor. Similar to the prior storm, most dissolved nitrogen entered the ponds as nitrate+nitrite, and removal efficiencies were variable. The large suspended sediment load at Pointe at Jamestown (256 kg) was reduced by 83% prior to discharge from the retention pond.

Table 2. Summary of removal efficiencies for two retention ponds during a storm on 5 July 2006.

Location	Rainfall (inches)	Parameter	Total In (g)	Total Out (g)	Removal Efficiency
Mulberry Place	0.88	Particulate P	24	1	98%
		Dissolved P	182	95	48%
		Ammonium	35	36	None
		Nitrate+Nitrite	778	560	28%
Pointe at Jamestown	1.85	Particulate P	168	56	66%
		Dissolved P	537	72	87%
		Ammonium	92	80	12%
		Nitrate+Nitrite	4,099	293	93%
		Suspended Sed	256,540	44,810	83%

DISCUSSION

Stormwater retention ponds are designed first to retain water and reduce peak volume discharges from developed watersheds with impervious surfaces. A secondary goal of stormwater retention is to improve water quality prior to discharge from the basin. We found that one of our monitored basins (Ironbound Village) did not meet this secondary goal of water quality improvement, as the pond was a source—not a sink—for suspended sediment during a storm. The other two basins, however, demonstrated variable but positive retention of suspended sediment and different forms of nitrogen and phosphorus. Stormwater retention can improve water quality through processes such as sediment settling and biological uptake in the basin.

The variation both among basins spatially and within basins temporally argues that the level of service of improved water quality cannot be predicted reliably. Further, retention ponds in James City County are constructed to hold a 1-year storm for 24 hours. None of the storms we measured approached the intensity or duration of a 1-year storm, nor did stormwater retention times approach 24 hours. We are concerned that the engineering solutions for these ponds overestimate retention times and thus overestimate water quality improvements. So many variables must influence pond performance that one cannot easily predict the downstream impacts resulting from the runoff of any single storm (Kumar Behara *et al.* 1999). To this end, Strecker *et al.* (2001) have recommended that effluent quality—not removal efficiency—should be the metric for determining water quality improvement.

This conclusion of unpredictable basin performance is disconcerting given that stormwater retention ponds are an accepted and expected component of many land development plans. In James City County alone, the number of permitted ponds has reached 500, but engineering designs have not improved dramatically, and humanpower available to oversee pond construction and monitor pond performance is practically non-existent. Because many of these

ponds are located adjacent to headwater streams, a failure to improve water quality locally gets transmitted and compounded with runoff from other ponds in the drainage area. Knowing where ponds occur and their potential for negative impacts downstream, a higher standard of pond design and performance is required if we ever expect to blunt the flow of non-point source pollutants from developing watersheds.

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**PILOT STUDY OF HEAVY METAL ACCUMULATION IN SEDIMENTS IN A
WETLANDS DESIGNED TO TREAT ACID MINE DRAINAGE:
PRELIMINARY RESULTS**

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ABSTRACT

The campus of the University of Virginia's College at Wise was mined from the 1960's through the 1980's. As a result, the campus has been affected by acid mine drainage. The College constructed a passive wetlands treatment system to mitigate acid mine drainage. The system has a design lifespan of seven to thirty years after which sediment must be removed for the system to continue to work properly. The presence of metalliferous shales associated with coal beds provides a potential source for heavy metals that may accumulate in wetland sediment. It is possible that low pH fluids associated with acid mine drainage may enhance metal movement, resulting in concentration of metals in sediments accumulating in the wetland treatment system. Initial results of sediment collection and digestion indicate the presence of elevated levels Pb, Zn, Cd, Co, and Ni. Levels of these of these metals are elevated by factors ranging from 2.5 for Ni to 5 for Co. In addition, samples taken from direct contact with concrete channels exhibit a distinct enrichment in Fe, Ni, and Zn, indicating that these species may be concentrated in open channels. Furthermore, preliminary data indicate the presence of a source of iron near pond four, likely from an acid mine drainage seep that has not been treated.

ANALYSIS OF LOW PRESSURE PROPAGATION IN PLUMBING SYSTEMS

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KEY WORDS: contaminant intrusion, pressure transients, plumbing systems, water hammer

ABSTRACT

America's water distribution infrastructure system is old and deteriorating. A water system with its myriad of appurtenances including pumps and valves and tanks is susceptible to hydraulic transients resulting in high and low pressure waves alternatively passing through the network. While both kinds of pressures structurally tax the already weak system, there is copious evidence to intrusion of contaminants into the drinking water pipes from the pipe exterior environment as the low pressure is unable to withstand the high pressure push. These contaminants are bound to enter into the house as the home plumbing system is a passive recipient from the water main. The major municipal system is readily recognized as a vast infrastructure system of nearly 1,409,800 km of piping within the United States, the minor plumbing system that is at least 5 to 10 times larger is generally not well addressed. An understanding of the pressure and flow pattern within a plumbing system should lead to a safer design not only within the house but also better design and maintenance practices for the municipal system.

INTRODUCTION

The growth in bottled water consumption, point of use water treatment including various kinds of filters, distillation, ion exchange, and reverse osmosis indicates citizen concern regarding the quality of the supplied water. However, cost advantage of the public water supply, maintenance problems related to the point of use devices and relative marginal improvement in water quality support the centralized public water supply. In this paper, we address the problem of intrusion of contaminated water from the exterior surrounding and backflow into the distribution piping through fracture cracks in the pipes, leaky joints, faulty seals, submerged air vacuum valves, and cross connections. It was widely believed that because a drinking water distribution system is pressurized the water can only come out; however, there is evidence that pump trips, opening and closing of fire hydrants, valve closures, pipe breaks, sudden change in demand and resonance, and malfunctioning of valves can induce significant transients that lead to a lower pressure within drinking water piping and a greater external pressure can result in intrusion. Tests of surrounding soil and pipe specimens from repair locations show the presence of pathogens. In 2000 alone in the U.S. 6,988 water systems affecting about 10.5 million people violated drinking water standards for microbial standards.

Le Chevallier *et al.* (2003) define intrusion as the specialized backflow situation in which nonpotable/contaminated water from the environment outside of the distribution piping flows into the pipe through available opening. Kirmeyer *et al.* (2001) and Friedman *et al.* (2004) point out physical mechanisms grouped by *transitory contamination* due to low pressure propagation in the system permitting a push-through of contaminants from the exterior surroundings with a higher pressure; *cross connection* between potable water system and a source that can potentially introduce contaminants into the potable water; and *pipe break, repair and installation* that expose the distribution system to the externalities as routes of entries. Storage facilities both covered and uncovered, purposeful contamination, growth and resuspension serve as additional sources for pathogen intrusion. Karim *et al.* (2003) reported bacteria and viruses in 66 soil and water samples collected next to drinking water pipelines in eight utilities in six states. Friedman *et al.* (2004) documented intrusions and low pressures of the order of negative 10 psi (gage). They also emphasize that the intruded contaminant is not re-extruded out of the pipe but a portion of it is carried downstream. Distribution mains downstream of pumps, high elevation areas, low static pressure zones, areas far away from overhead tanks, segments of pipes upstream and downstream of active valves in high flow areas are the most susceptible for low or negative pressures. Locations with frequent leaks and breaks, high water table regions, flooded air vacuum valve vaults, and high risk cross connections have the highest potential for intrusion. Gullick *et al.* (2004) observed most surge events as the result of pump operations and outages.

In this paper we analyze the water hammer related transients within a plumbing system and possible contamination at the household level. As the home plumbing system is a passive recipient from the water mains, if there is street level contamination it is bound to enter into the house. We like to explore for varying street level boundary conditions and the conditions inside of the house, whether low pressures can be induced within the plumbing system.

METHODS

The analysis of plumbing systems differs markedly from the analysis of municipal systems. It is typical practice to treat the municipal systems as steady flow systems and the continuity and energy equations are solved; however, in plumbing systems because the demands last for hardly a few minutes transient analysis is more appropriate and the continuity and momentum equations will have to be solved. We are using WHAMO (Water Hammer and Mass Oscillation) package from the U.S. Army Corps of Engineers that can solve the transient flow problem. WHAMO is designed for hydropower systems to analyze transients induced by pumps, turbines, and turbine pumps involving valves, governors, reservoirs and surge tanks. In this study we are solving for pressure and flow patterns in small diameter pipe (0.75 to 1.5 inches) networks involving valves, a pump, and a tank. The purpose of WHAMO application is to improve the unsteady phase design of an experimental plumbing rig before we actually construct it. In addition to the unsteady phase, it is also crucial to have an understanding of the steady state of the system.

Background hydraulics

The water hammer is a transient flow phenomenon introduced in pipe flow systems by suddenly obstructing the flow. As a consequence, there is a pressure rise and fall and the pattern is repeated until the transients decay. For completeness, we present the water hammer equations here as

Continuity equation:
$$\frac{\partial p}{\partial t} + V \frac{\partial p}{\partial x} + c^2 \rho \frac{\partial V}{\partial x} = 0$$

Momentum equation:
$$\frac{\partial V}{\partial t} + V \frac{\partial V}{\partial x} + \frac{1}{\rho} \frac{\partial p}{\partial x} + g \sin \alpha + \frac{f}{2D} V|V| = 0$$

in which: p = pressure, V = velocity, c = wave speed, ρ = density, g = acceleration due to gravity, α = angle of inclination of pipe, f = friction factor, D = diameter, x = spatial dimension, t = time. These equations have to be solved for a network incorporating suitable interior boundary conditions for such appurtenances as valves, pumps, and junctions where several pipes join and external boundary conditions for street level lateral, tanks, and faucets. Wylie and Streeter (1993) provide detailed accounts of the solution methodology. The solution of these equations yields the pressure, $p(x, t)$ and velocity, $V(x, t)$ as functions of x and t with x – dimension is taken along the length of the pipe. The pressure can be high positive and negative and the velocity can be negative indicating flow reversal.

Plumbing Rig simulation

The schematic of the system is shown in Figure 1. Each node is identified with a number and the pipes are listed as C1, C2... and C6. There are seven nodes and six pipes. Nodes 100 and 200 are reservoirs with 5 and 5.1 feet of head respectively. Pipes are copper L type and 1.5 and 0.75 inches in diameter. We are using 1 hp centrifugal pump (P1) and the valve (V1) with minor loss coefficient of 10. An overview of the system is summarized by pipes in Table 1 and by nodes in Table 2.

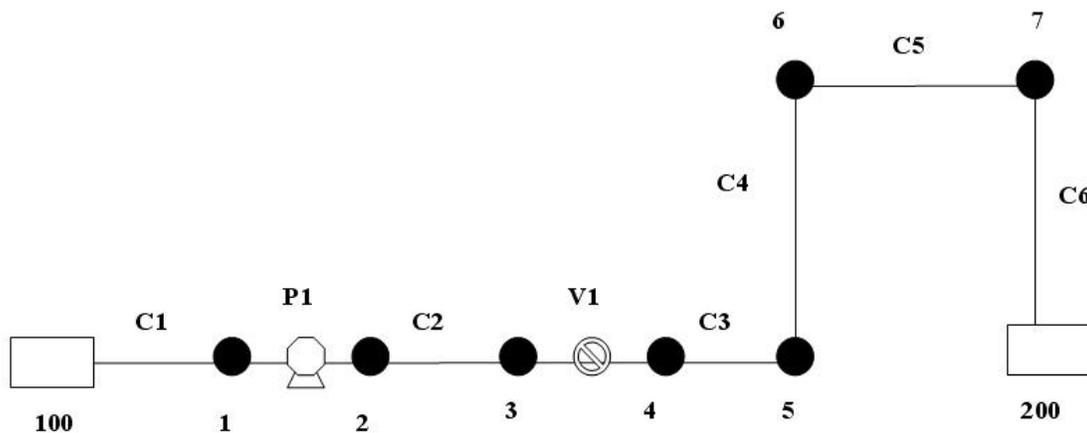


Figure 1. System Layout

Table 1. Pipe Properties

Conduit	Length (feet)	Diameter (feet)	Friction Factor
C1	3.00	0.13	0.023
C2	6.00	0.06	0.021
C3	2.00	0.06	0.021
C4	7.00	0.06	0.021
C5	11.00	0.06	0.021
C6	2.10	0.06	0.021

Table 2. Node Properties

Node	Elevation (feet)
100	5.00
1	0.00
2	0.00
3	0.00
4	0.00
5	0.00
6	7.00
7	7.00
200	5.10

RESULTS AND DISCUSSIONS

Steady state results are shown in Table 3. It is seen that Whamo, EPANET (Throttle valve and minor loss case), and analytical solutions are yielding very close results. One of the EPANET simulation run (*EPANET_TCV*) was normal throttle control valve (V1) with minor loss coefficient 10 and the other simulation (*EPANET_MinorLoss*) is the short length (0.001') of pipe with loss coefficient 10. We wanted to check the sensitivity of each appurtenance on the outcome. To verify the simulation results, analytical solution was obtained using energy equation for each node. Batterton (2005) has shown how the steady state momentum equation can be reduced to the steady state energy equation to explain the identical solutions even though there are small discrepancies due to numerical solvers.

Table 3. Steady State Comparison

Node	Whamo (feet)	EPANET _TCV (feet)	EPANET _MinorLoss (feet)	Analytical Solution (feet)
100	5.00	5.00	5.00	5.00
1	4.92	4.92	4.92	4.92
2	49.06	48.04	48.04	49.36
3	44.50	43.60	43.60	44.76
4	21.82	21.38	21.38	21.95
5	20.30	19.90	19.90	20.42
6	14.98	14.71	14.71	15.05
7	6.61	6.57	6.57	6.62
200	5.01	5.01	5.01	5.01

Figure 3 shows the pressure behavior when the valve is shut off (while pump is normally running) at 7 seconds respectively. Nodes 6 and 7 show a negative pressure of -2 psi.

C Experimental Setup

WHAMO Simulation Time History

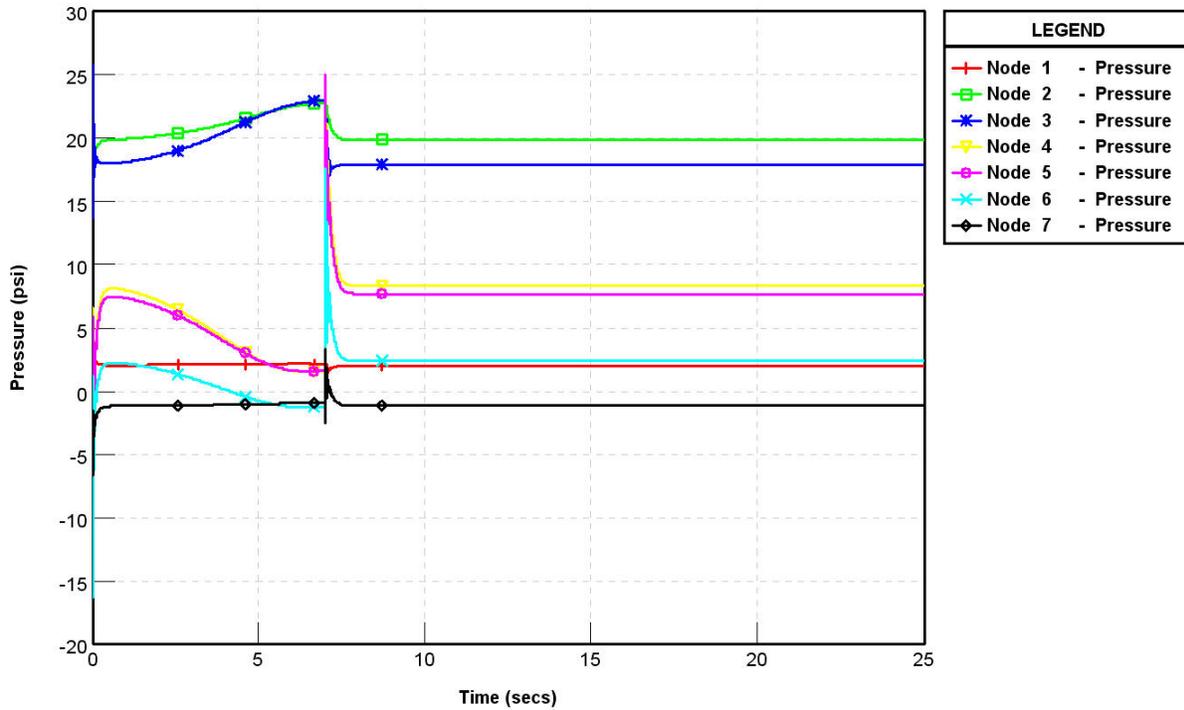


Figure 3. Valve Shut Off at Time t = 7 Seconds

SUMMARY

In this paper, an analysis related to: (1) how a low pressure wave moves through the plumbing system as the result of a street level transient and (2) how a transient triggered within a house can impact the street lateral with a possible suction effect is presented. An understanding of the pressure and flow pattern within a plumbing system should lead to a safer design not only within the house but also better design and maintenance practices for the municipal system.

ACKNOWLEDGEMENT

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MANGANESE SCALES AND BIOFILMS IN DRINKING WATER DISTRIBUTION SYSTEMS

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KEY WORDS: manganese oxidation, PVC, iron, biofilm

ABSTRACT

The possible impact of PVC and iron pipes on drinking water quality has been evaluated and analyzed in the distribution system of the water treatment plant of “La Concepción” in Tegucigalpa, Honduras. Black water problems have been reported in the distribution system of Tegucigalpa due to biogeochemical cycling of manganese (Mn). Higher total Mn concentrations (35-fold more) and more black water problems were detected in PVC compared to iron pipe. Analysis of the pipe surfaces using Scanning Electron Microscopy showed that Mn was incorporated into the iron pipe and thus not readily removed by water flow. Mn was easily dislodged from the PVC pipe surface. Manganese-oxidizing bacteria were isolated from both PVC and iron pipe biofilms obtained from the distribution system. Eighty percent (80%) of isolates from the PVC biofilm were capable of Mn-oxidation while only 35 % of isolates from the iron pipe biofilm. This research demonstrates the importance of the different interactions between water and the infrastructure used for its supply in producing potable drinking water.

INTRODUCTION

The interactions between water and the infrastructure used for its supply are fundamental in producing safe drinking water. Subtle reactions between water and the different materials used for its transport can result in alterations that affect the ultimate quality delivered to consumers. A clear example of this situation is the presence of manganese in drinking water. A secondary maximum contaminant level of 0.05 mg/L has been established for manganese by the U.S. Environmental Protection Agency. Manganese can produce aesthetic problems such as water discoloration, fouling and staining on plumbing fixtures caused by the oxidation of soluble

manganese in distribution systems (Sly *et al.* 1990). When manganese is oxidized from its soluble (Mn^{+2}) to its insoluble form (Mn^{+4}) it forms a black precipitate which causes water color problems. Oxidation of soluble manganese to its insoluble form with the use of a strong oxidant and then processing of such solids through solid-liquid separation is a common treatment method for manganese removal from drinking water (American Water Works Association 1999, Knocke *et al.* 1991). Inadequate treatment of manganese at the water treatment plant can cause soluble manganese to enter the distribution system where it is oxidized to its insoluble form either by residual concentrations of chlorine or by bacteria, depositing these oxidized solids on the pipe surfaces (Hasselbarth 1972, Sly *et al.* 1990).

High levels of MnO_2 solid were first detected in the distribution system of Tegucigalpa in April and May of 1998, and have continued to cause occasional severe water color problems, leading to “black water” at home taps, which brought complaints. In Honduras, PVC constitutes 79% of main lines, and 55% of house plumbing, followed by galvanized iron (14% in main lines, and 33% in house plumbing). According to information provided by the drinking water consumers, black water problems were more noticeable where PVC pipe was used for distribution system mains and house plumbing. Fewer complaints were reported in areas where iron pipe was used for distribution system mains and household plumbing. Due to water quantity problems, the water service is partially suspended in certain sectors of the distribution system at prescheduled times allowing intermittent flow conditions to occur. The neighborhoods sampled for this project received 16 hours of continuous flow and 8 hours of no flow.

The overall goal of this study was to investigate the effect of piping materials -PVC and iron- on drinking water quality for a water supply system constantly fed by Mn (II). The specific objectives of the project were: evaluate the possible influence that PVC and iron pipes could have on the concentration of manganese in drinking water; analyze the effects caused by the presence of manganese in PVC and iron pipe surfaces; and analyze the possible influence of microbiological biofilms on manganese oxidation in drinking water systems.

METHODS

Representative sections of the distribution system of the reservoir and water treatment plant of “La Concepción” in Tegucigalpa, Honduras were sampled during July and August of 2004. Four water sample replicates ($n = 4$) were obtained for each of the following locations: water treatment plant influent, water treatment plant effluent, PVC (15 years old) and iron (30 years old) pipes in the distribution system mains. The neighborhoods selected for sampling were relatively close to the water treatment plant. The chosen sampling point for PVC and iron pipe distribution system pipe mains were located at a distance of approximately 7 km and 9km away from the water treatment plant respectively. A key characteristic of these neighborhoods is that they received an intermittent service, for which water was provided from 3pm - 7am (meaning that the pipes had no-flow for 8 hours, from 7am – 3pm). The term “first flush” will refer hereafter to the water flow in the distribution system resulting from the reestablishment of the service after 8 hours of interruption. The distribution system pipe mains were sampled twice, at the exact time when the service was reestablished (first flush, approximately 3pm) and at peak hour flow conditions (around 6:30pm and 8:30pm).

Duplicate water samples were collected in 60 mL acid washed polyethylene bottles for metals analyses. One sample was immediately filtered on-site using a 0.45 μ m membrane to remove suspended solids or particles. Both collected samples for metal analysis were acidified with 1:1 HNO₃:H₂O for conservation and were stored at room temperature until they were analyzed. Soluble and total concentrations of Mn were measured using a JY Ultima Inductively Coupled Plasma – Emission Spectroscopy (ICP-ES) instrument according to Standard Method 3120B (American Public Health Association 1998). Portable equipment was used to measure the following parameters: dissolved oxygen using a digital titrator (HACH methods 8332); pH by pHHydriion tape; and total chlorine by means of a DR/4000 HACH Spectrophotometer (DPD – total chlorine method). PVC and iron pipe samples collected at the sampled locations of the distribution system. Inorganic elements in the interiors of the pipe surfaces were analyzed using a Leo 1550 field emission scanning electron microscope (FE-SEM) equipped with an IXRF electron dispersive spectroscopy (EDS). All elemental compositions were reported as % weight contents. The sample surfaces were plated with gold palladium at 9 nm thickness, and because these elements are rare in potable plumbing systems, gold and palladium were not measured in the surface analysis. After inspecting the surface with the SEM, surface sites were chosen at random for elemental analysis in an attempt to identify differences of the surface.

For microbial enumeration of Mn-oxidizing bacteria, dilution series of the biofilm suspensions were prepared in 5 ml sterile distilled water (10^{-1} to 10^{-5}) and 0.1 ml in duplicate was spread on the Manganese-selective Agar (per liter of 10 mM HEPES buffer (pH 7.4): 0.001 gm FeSO₄·7H₂O, 0.15 gm MnSO₄·H₂O, 2 gm Peptone, 0.5 gm Yeast Extract, and 15 gm agar). The media was incubated at 30° C for 3-5 days and the number of total colonies and individual colony types were counted. Colonies were examined on Mn-selective medium for deposition of dark (oxidized) Mn. The number of total microorganisms and each colony type was calculated per cm² of biofilm sampled for each of the agar media. Single isolated colonies of each type were picked and inoculated into 10 ml of Mn-oxidation broth (per liter of 10 mM HEPES buffer (pH 7.4): 0.001 gm FeSO₄·7H₂O, 0.2 gm MnSO₄·4H₂O, 2 gm Peptone, and 0.5 gm Yeast Extract) in a 125 ml side arm flask and incubated at 30° C at high aeration (120 rpm). Then 0.2 ml of culture (early stationary phase) were mixed in a microcentrifuge tube with 1 ml of 0.04 % Leucoberbelin Blue I (Aldrich 43,219-9) in 45 mM Acetic Acid and immediately centrifuged (10,000 x g) for 5 min to pellet the cells. The supernatant was transferred to a 1 ml spectrophotometer cuvette and absorbance was measured at 620 nm for formation of oxidized Mn [*i.e.*, Mn(IV)].

RESULTS

The results for the water quality parameters of interest are presented in Table 1. Manganese was mainly soluble in the influent and the obtained mean concentration was higher than the established secondary MCL (0.05mg/l) by the U.S. EPA. The fact that Mn was still mainly soluble at the effluent of the water treatment plant indicates that there was not well removed. Very high concentrations of manganese in particulate form were then observed at the distribution system, especially for first flush conditions. Dissolved oxygen was found at levels above 6 mg/L throughout, which are enough to have an effect on different chemical and microbiological processes. Residual chlorine decayed rapidly in the distribution system of Tegucigalpa as a function of distance.

Samples of PVC and iron pipes from the distribution system were analyzed by SEM. Three different scale layers were analyzed for the iron pipe: the bottom section of the iron pipe itself, an intermediate rusty brown layer between the surface scale and the iron pipe material, and a white layer which was the surface scale in direct contact with the water. Another three different scales were analyzed for the PVC pipe sample: the bottom part of the PVC pipe, an intermediate white layer in between the surface and bottom PVC interface, and a brown layer which was the surface scale in contact with the water. Acid digestion of solids scraped from the PVC and iron pipe surfaces was subjected to elemental analysis. Table 2 shows a summary of the obtained results for the SEM analysis on a weight percentage contents basis of selected elements that were determined for the pipe samples and scales.

Table 1. Concentrations (mg/l) for Water Quality Parameters of Interest

Concentration (mg/l)				
Sample	Manganese		Chlorine	Dissolved Oxygen
	Total	Dissolved		
Plant Influent	0.282 ± 0.012	0.259 ± 0.011	BDL ³	N.A. ⁴
Plant Effluent	0.261 ± 0.036	0.254 ± 0.034	1.250 ± 0.06	9.3
PVC (First Flush)	15.732 ± 10.323	0.014 ± 0.009	0.550	8.2
Iron (First Flush)	0.743 ± 0.471	0.018 ± 0.004	0.375 ± 0.007	7.1
PVC (Continuous)	0.821 ± 1.469	0.068 ± 0.025	0.610 ± 0.038	10.5
Iron (Continuous)	0.036 ± 0.012	0.021 ± 0.006	0.310 ± 0.061	8.1

¹ BDL = Below Detection Limit.

² N.A. = Not Analyzed

Microorganisms were recovered from biofilms present on both iron and PVC pipe sections. The obtained results from the analysis of manganese-oxidizing bacteria are presented in Table 3. Higher numbers of colony-forming microorganisms were recovered from Iron pipe (30-fold more) compared to PVC pipe sections. The majority of isolates from the PVC biofilm (8 of 10, 80 %) were capable of Mn-oxidation. In contrast only 35 % (11 of 31) of isolates from the iron biofilm sample demonstrated Mn-oxidation.

DISCUSSION

Higher concentrations of total manganese were obtained in the water samples collected from the PVC pipe compared to the iron pipe. The difference in manganese release in the pipes was likely due to differential precipitation and interaction of metal species with the pipe material. This was further investigated by examining the chemical compositions of the scales that formed in the pipes. PVC material is non polar and does not exchange electrons with manganese attached to its surface for which a superficial scale of manganese precipitate was formed. This surface scale

in the PVC pipe was brown colored and had a gelatinous texture when the pipe had water flowing on it and a dry texture which looked like a broken egg shell during first flush conditions. Water motion creates a shear stress at the fluid - pipe material interface causing the release of manganese present at the surface scale. More manganese solids were observed in water samples obtained from both PVC and iron pipes for first flush compared to continuous flow due to the fact that water velocities and turbulence were higher when the service was reestablished after being interrupted for 8 hours. Release of manganese solids and water color problems were more critical in the PVC pipe for first flush because the exerted shear stresses scraped its surface scale while it had a dry texture. However, significant concentrations of manganese in particulate form were still released for continuous flow from the PVC pipe, where flow conditions were supposed to be more stable and the surface scale wetted to have a gelatinous texture. According to such results, it can be observed that the surface scale of the PVC pipe was more susceptible to manganese release when it had a dry (characteristic of no - flow conditions) rather than a gelatinous (characteristic of continuous flow conditions) texture.

Table 2. Weight Percentage Content of Relevant Elements Found on the Pipe Sample Surfaces using Scanning Electron Microscopy

%Weight Composition						
	PVC			Iron		
Element	Pipe Material	White Scale (Intermediate)	Brown Scale (Surface)	Pipe Material	Brown Scale (Intermediate)	White Scale (Surface)
C	38.99	BDL ¹	BDL	0.02	BDL	BDL
O	2.24	52.35	49.63	29.30	49.71	51.64
Mn	0.11	0.16	6.12	0.06	0.12	3.48
Fe	0.10	1.19	1.42	63.47	8.14	6.33
Zn	0.20	0.14	0.35	1.25	0.29	0.74

¹ BDL = Below Detection Limit

The iron pipe is a rich source of electrons. According to the results shown on Table 2, the iron pipe surface scale contained both iron and manganese. The appearance of the surface scale from the iron pipe samples was that of a corroded surface with several tubercles. Manganese attached to the surface scale of the iron pipe exchanges electrons with other elements with which it can perfectly couple, such as iron. This interaction allows manganese to be incorporated to other physical chemical processes, such as the case of corrosion, for which a layer formed by manganese precipitates was not clearly identified on the surface scale of the iron pipe as it was on the PVC pipe. For first flush conditions there was a significant release of manganese but, due to its roughness and hardness, the iron pipe surface was not easily scraped by water motion shear stress for continuous flow conditions where there was not a significant release of either iron or manganese. For this reason water color problems were less observed by costumers located on sections of the distribution system served by iron compared to PVC pipe.

The fact that manganese-oxidizing bacteria have been isolated from both PVC and iron pipes from the water system of Tegucigalpa is of great significance. This result suggests that chemical oxidation is not the only factor affecting manganese transformations; but microbiological oxidation should also be considered. Other studies have also shown microbiological oxidation of

manganese in drinking water distribution systems (Sly *et al.* 1990, Murdoch *et al.* 2000). Increased knowledge of manganese and metal cycling in distribution systems is critical if utilities are to control these phenomena. Because water in distribution systems is simultaneously subjected to chemical and microbial mediated redox reactions, separating these two mechanisms to determine their relative importance is a major challenge. Further research should be performed in order to account for such differences.

Table 3. Number of Total and Mn-oxidizing Microorganisms in Iron and PVC Biofilms

Source of Biofilm	Total Microorganisms (CFU/cm ²)	Number of Isolates Tested	Mn-Oxidizers Number (Percent)
Iron Pipe	2.6 x 10 ³ /cm ²	31	11(35%)
PVC Pipe	8.8 x 10 ¹ /cm ²	10	8(80%)

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RE-GROWTH OF *LEGIONELLA PNEUMOPHILA* IN POTABLE WATER DURING STORAGE IN DOMESTIC WATER HEATERS

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KEY WORDS: *Legionella pneumophila*, microbial re-growth, domestic plumbing

ABSTRACT

A reproducible means to study amplification of *Legionella pneumophila* under conditions found in domestic water heaters has been developed using synthesized potable water and ozonated humic acid as organic matter. Eight realistic nutrient levels and four disinfectant conditions were tested using this approach, and observed levels of *L. pneumophila* were in the range of those encountered in Legionnaires' disease outbreaks. When 0.25 mg/l chlorine or 0.40 mg/l monochloramine were present, *L. pneumophila* was inhibited. High levels of *L. pneumophila* were not developed without the addition of ozonated humic acid or phosphate. Future studies will determine the levels of organic matter and phosphate required for *Legionella* growth, and effects of *Hartmannella vermiformis*, iron, temperature, pH, and amino acids.

INTRODUCTION

Domestic water heaters have three characteristics that can allow bacteria to proliferate: temperature, biofilms, and low disinfectant residuals. In the United States, the Department of Energy and Consumer Product Safety Commission recommend that water heater temperatures be kept below 120°F (49°C) to save energy and prevent scalding (DOE, CPSC). However, temperatures below 140°F (60°C) do not effectively control microbial re-growth, particularly since thermal stratification can occur which results in dramatically lower temperatures in the bottom of the water heater (Spinks *et al.* 2003). At temperatures lower than 60°C, biofilms can also form in the water heater, allowing complex microbial flora to thrive (Keevil *et al.* 1995). Water stored in domestic water heaters has a longer detention time than domestic cold water, chlorine decays faster at warmer temperatures, and the bottom of the heater often contains sediment—the net result is that relatively little or no disinfectant is present to control re-growth.

One pathogen that can reside in water heaters is *Legionella pneumophila*. *L. pneumophila* is an opportunistic pathogen and the causative agent of Legionnaire's disease, a severe form of pneumonia that infects 8,000 to 25,000 people per year in the United States (CDC 2005, OSHA).

Infection requires inhalation or aspiration of contaminated water, which can occur in showers or hot tubs. A review of the literature suggests that about 25-39% of water heaters worldwide contain *Legionella* (Alary and Joly 1991, Hedges and Roser, 1991, Zacheus and Martikainen, 1994). *L. pneumophila* require amino acids for growth and thrive within amoeba such as *Hartmannella vermiformis* that can be found within biofilms (Wadowsky *et al.* 1991).

MATERIALS AND METHODS

Thirty-two disinfectant and nutrient combinations were used to simulate the water in a typical water heater. A synthetic base water was made and contained 0.180 mg/l total organic carbon (TOC), 0.010 mg/l Sodium Phosphate Dibasic (Na₂HPO₄), 9.7 mg/l Potassium Nitrate (KNO₃), 57 mg/l Sodium Bicarbonate (NaHCO₃), 39 mg/l Magnesium Sulfate (MgSO₄), 0.73 mg/l Aluminum Sulfate (Al₂(SO₄)₃), 26 mg/l Sodium Silicate (NaSiO₂), 25 mg/l Calcium Sulfate (CaSO₄), 21 mg/l Calcium Chloride (CaCl₂), and 5 µg/l Ethylenediaminetetraacetic acid, iron(III) sodium salt hydrate (C₁₀H₁₂FeN₂NaO₈) as Fe. Ozonated Aldrich humic acid was used for organic matter. Ozone was dosed into the humic acid so that 87% of the color was destroyed. Eight modifications to this base water were studied and are shown in Table 1.

Table 1. Nutrient Modifications to Base Water

Condition #	Nutrient Modification
1	0 TOC
2	¼ TOC
3	4 times greater TOC
4	1/30 KNO ₃
5	No changes
6	3 times greater Na ₂ HCO ₃
7	0 Na ₂ HPO ₄
8	20 times greater Na ₂ HPO ₄

Chlorine (0.25 mg/l) and chloramines (0.40 mg/l) were used for disinfectants. No disinfectant and ammonia (1.0 mg/l) were also studied to simulate what would occur in homes after the residual disinfectant has decayed. That is, the no disinfectant condition simulates what would occur after chlorine decays, whereas the condition with ammonia simulates what would occur after 4.0 mg/l monochloramine has decayed. Thirty-two 118 ml glass bottles were inoculated with 50 ml of water from a domestic water heater, 30 µl of sediment-slurry from a small outdoor pond on the Virginia Tech campus in Blacksburg, Virginia, and *Legionella pneumophila* (ATCC 33152) to create a concentration of 29,200 cfu/ml *L. pneumophila* in each bottle. The bottles were kept in the dark and incubated at 37°C for three days to simulate the conditions at the bottom of a water heater. The solution in each bottle was then disposed of and replaced with 100 ml of one of the nutrient and disinfectant combinations. The bottles were again kept in the dark and incubated at 37°C. The water was disposed of and replaced twice per week to maintain disinfectant residuals and simulate typical conditions of a domestic water heater.

Legionella was quantified using BCYE and DGVP agar (Wadowsky and Yee 1981). An acid pretreatment containing 0.2M HCl and 0.2M KCl was reacted with the sample in a 1:3 ratio for 5 minutes before plating onto BCYE (adapted from Stout 1998). After 3-7 days of incubation at

37°C, colonies with morphology consistent to that of *Legionella* were streaked onto BCYE and 7% sheep blood agar (BAP). Growth on BCYE and not BAP was considered presumptive for *Legionella*. Heterotrophic plate counts were performed by plating the sample on R2A agar and incubating for 7 days at room temperature. Buffered saline gelatin was used to create necessary dilutions before plating. *Legionella* was not detected in the above tests after 9 months. Consequently, two inoculations of the Philadelphia-1 strain of *Legionella pneumophila* were used for each reactor.

RESULTS AND DISCUSSION

The results of this study demonstrated that a method for replicating *Legionella pneumophila* in synthetic drinking water has been developed. The levels of *L. pneumophila* recorded were within the range at which outbreaks have occurred. In the presence of 0.25 mg/l chlorine or 0.40 mg/l monochloramine, *L. pneumophila* was not present as shown in Figure 1. Once the chlorine or monochloramine has decayed, as demonstrated by the no disinfectant and ammonia conditions, respectively, *L. pneumophila* can grow at outbreak levels. This has severe implications for homes which receive water at the end of a distribution line or do not use enough water to maintain an adequate disinfectant residual.

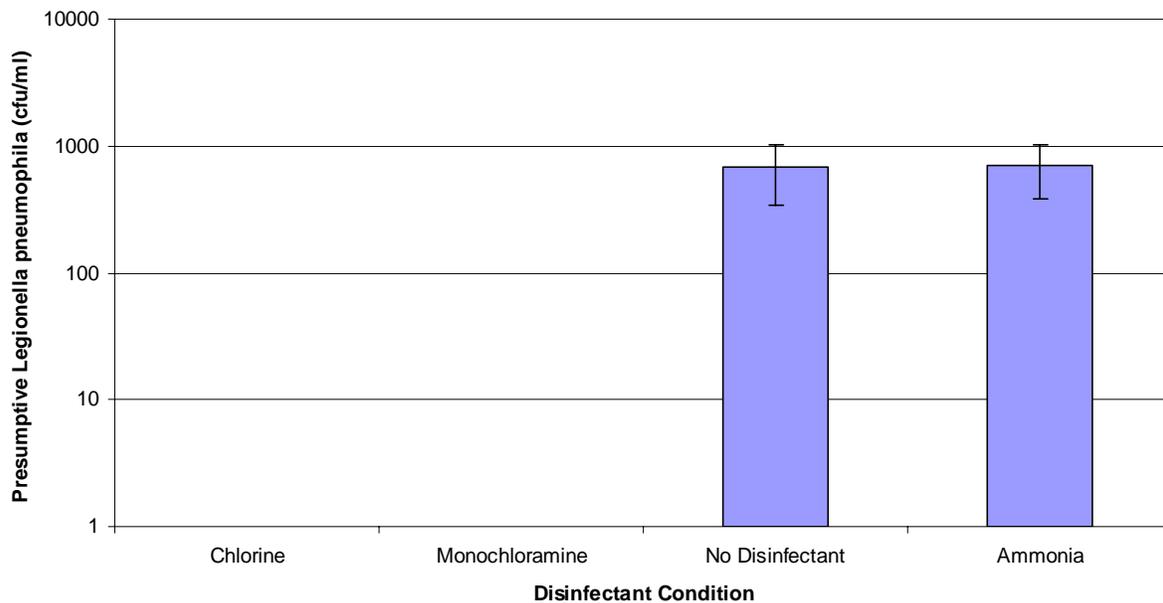


Figure 1. Average concentration of *Legionella pneumophila* for each disinfectant condition

When no added phosphate or ozonated humic acid was added, *Legionella pneumophila* recovered was over 100 times lower than the other conditions and did not reach outbreak levels, as shown in Figure 2.

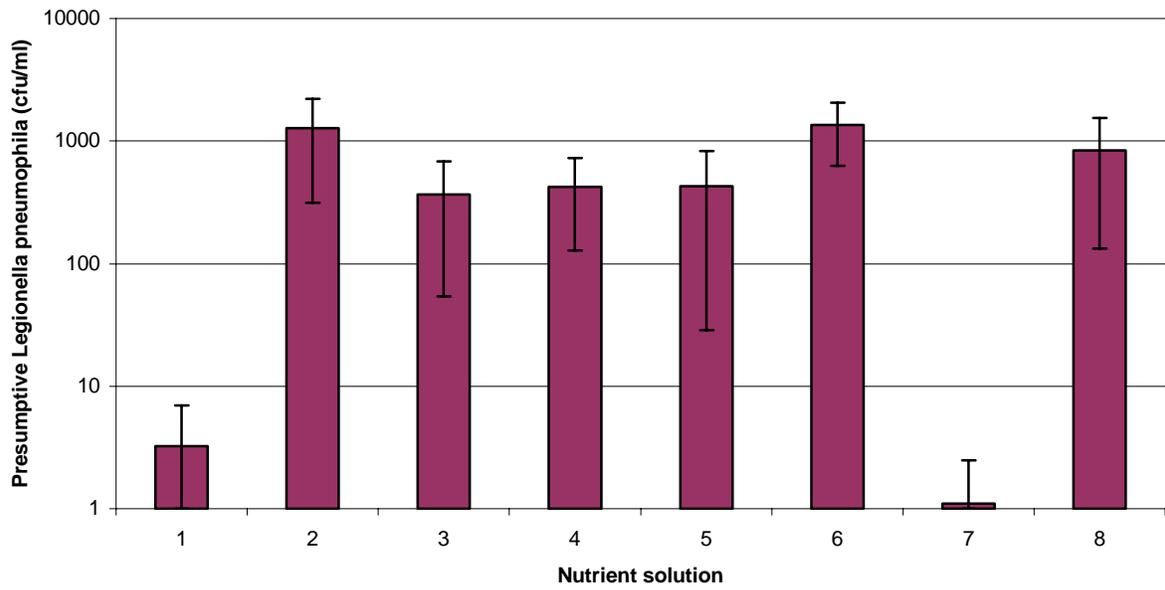


Figure 2. Average *L. pneumophila* concentration for nutrient conditions defined in Table 1

Figure 3 demonstrates that the overall heterotrophic bacterial population was not lower without added phosphate or ozonated humic acid as *L. pneumophila* was.

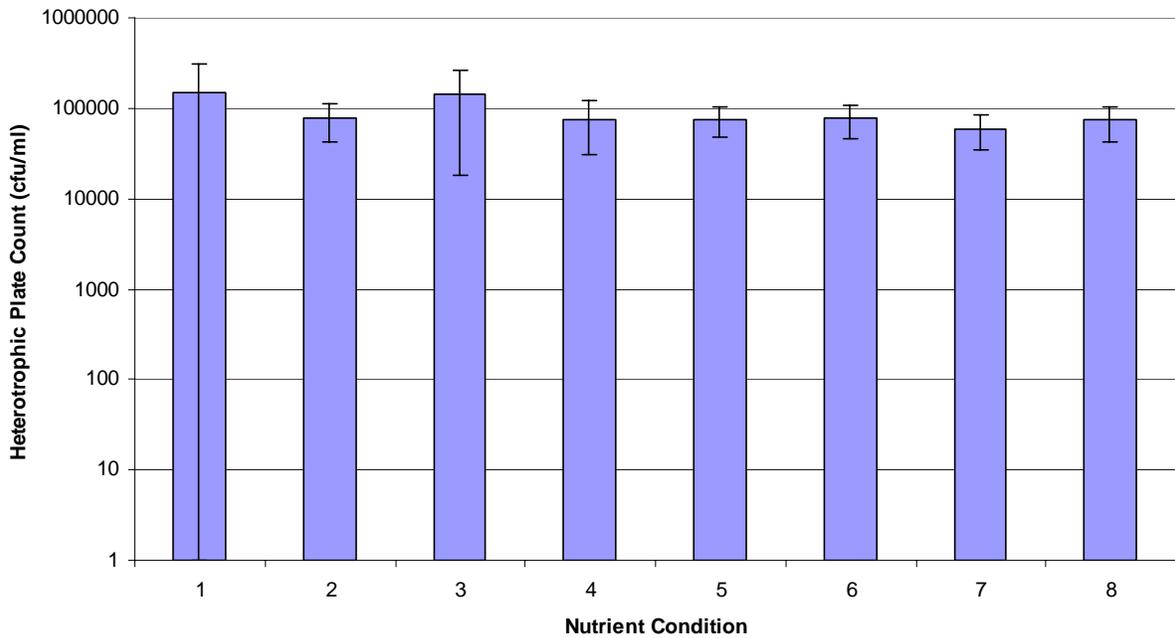


Figure 3. Average heterotrophic plate count for nutrient conditions defined in Table 1

CONCLUSIONS

The development of a synthesized potable water that sustains *Legionella pneumophila* growth is a critical step in studying factors that might lead to occurrence of Legionnaires' disease, since all prior growth media use unrealistically high or non-quantifiable levels of nutrients relative to drinking water. This success allows the relative impacts of drinking water treatment practice and water chemistry to be examined under oligotrophic conditions. Preliminary work demonstrates that phosphate and organic matter have a significant effect on *L. pneumophila* growth. Future studies will determine the level of organic matter required for growth and the effect of *Hartmannella vermiformis*, iron, temperature, pH, and amino acids.

ACKNOWLEDGEMENTS

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NITRIFICATION EFFECT ON LEAD AND COPPER LEACHING IN HOME PLUMBING SYSTEMS

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KEY WORDS: nitrification, lead, copper, home plumbing

ABSTRACT

The effect of nitrification on leaching of lead/copper was investigated under typical home plumbing configurations. Among four extremes of disinfectant conditions studied (free chlorine, chloramine, no disinfectant and free ammonia), nitrification occurred most strongly when free ammonia was applied. The pH decrease occurring in PVC pipes was correlated strongly to the extent of nitrification ($R^2 = 0.91$), whereas the pH decrease in lead, solder and brass containing rigs was partly correlated to nitrification ($R^2 = 0.2 - 0.5$). The lower pH increased lead and copper leaching with ammonia in lead containing rigs, indicating that nitrification can increase lead and copper release under some circumstances.

INTRODUCTION

Elevated lead and copper levels in drinking water have recently received increased scrutiny due to cases of childhood lead poisoning tied to potable water in Durham and Greenville, NC. Potable water leaving treatment plants almost invariably has low copper and lead levels, but lead and copper increase in water as it sits stagnant within home plumbing (*Federal Register 1991*). Copper pipe is the dominant material used to plumb buildings; lead pipe and leaded solder were in use until banned in about 1986, and leaded brass (up to 8% lead by weight) are still available for sale and installation. The most common use of lead pipe is the service line connecting the water main to home plumbing.

Lead can leach from brass, solder or lead pipe by itself, or lead corrosion can be driven by direct galvanic connections with copper (Reiber 1991, Dudi 2004). In such situations, the lead bearing material generally serves as the anode and is sacrificed by the copper:



and the cathodic reaction (*e.g.*, O_2 reduction) occurs on the copper surface:



The rate of attack on the lead material alone or via galvanic corrosion is influenced by numerous factors including water chemistry, disinfectant types (Dudi 2004) and bacterial growth.

Nitrification refers to the conversion of ammonia to nitrite and then nitrate by nitrifying bacteria (U.S. EPA 2005). Concern over nitrification in drinking water system is amplified because many utilities are switching to chloramine (formed by chlorine and ammonia) to comply with EPA

Stage 1 and Stage 2 Disinfectants and Disinfection By-Products Rule (D/DBPR) (U.S. EPA 2005). The free ammonia released during chloramine decay can serve as a nutrient source for nitrifying bacteria. Nitrification can cause decreasing pH and alkalinity, production of nitrite at levels of concern relative to drinking water guidelines, accelerating loss of chloramine disinfectant and increasing bacterial growth rates. Nitrification has been associated with practical problems encountered with lead in Washington D.C., Greenville, Durham N.C. and Ottawa (Douglas *et al.* 2004) after switching from free chlorine to chloramine, although the extent to which it contributed is uncertain. Elevated copper levels at the tap were also clearly tied to activity of nitrifying bacteria in Willmar homes (Murphy *et al.* 1997). Corrosion and metals leaching could be affected by nitrification through the reduced pH and alkalinity, and perhaps, through other impacts on bacteria growth and microbially induced corrosion (McNeill and Edwards 2001).

The objective of this study is to investigate the effects of nitrification on leaching of metals using plumbing configurations commonly found in homes.

MATERIALS AND METHODS

Pipe rig setup

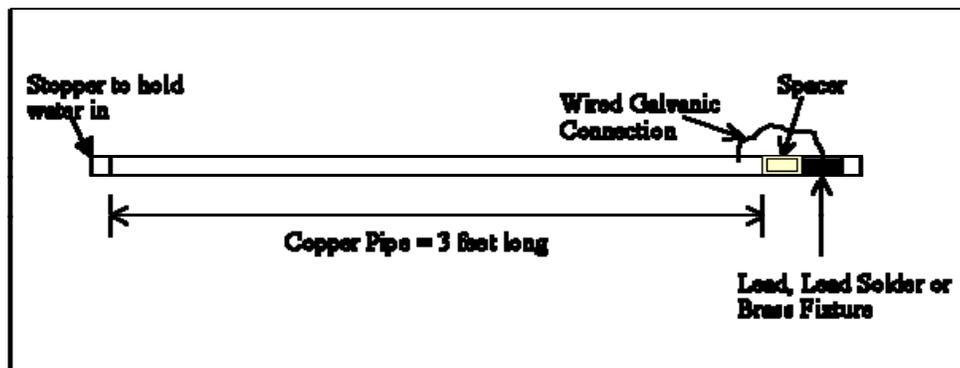


Figure 1. Rig setup to explore galvanic corrosion of copper pipes connected with lead-based materials.

The basic experimental rig simulates a connection between copper pipe and a lead bearing material such as brass, lead solder or lead pipe (Figure 1). The copper pipe is a 3' (length) \times $\frac{3}{4}$ " (diameter) section that is electrically connected to lead-based material (lead, tin solder or brass fixture) via an external grounding wire. A plastic ball valve spacer is also placed between the two materials to allow separate sampling of the two sides. As a control and basis for comparison, pipe rigs were also setup using PVC pipes of the same length.

Water Chemistry

Blacksburg, VA tap water was used as the base water for this study. The chloramine residual in this tap water was first removed by adding free chlorine to achieve breakpoint chlorination (Snoeyink and Jenkins 1980). This water was then modified to achieve four extremes of disinfection that might be encountered in potable water systems including: no disinfectant, free chlorine, chloramine and free ammonia. Water with free chlorine and chloramine represents the

water received by homes near the treatment plant, where disinfectant decay is negligible. No disinfectant and free ammonia represent the extreme conditions where free chlorine and chloramine are completely decayed during long transportation in distribution system or after prolonged stagnation, as might happen in homes. Free chlorine and chloramine were targeted at 1.5 mg/L and 4 mg/L, respectively. One mg/L ammonia was added to simulate the condition when chloramine was completely decayed. The pH of all test waters was adjusted to 7.8 ± 0.1 , which is the target pH of finished water for Blacksburg water. Each water quality was tested in duplicate, so there were 4 pipe materials (copper-lead, copper-solder, copper-brass, and PVC) \times 4 disinfectant types \times 2 duplicates = 32 tests. Water was changed in the pipes every 3.5 days (twice a week), using a “dump and fill” protocol.

After 11 months at the above condition, no nitrification was detected in any of these pipes, so the water preparation method was modified to encourage nitrification. Specifically, 500 ml extra test water was prepared for each condition, and this water was allowed to stay stagnant until the next water change, and then mixed with the next batch of new test water (Figure 2). The water change schedule was modified to five times each week (24 hour stagnation time during the week) during the intensive sampling herein in order to replenish nutrients more frequently and to better maintain disinfectant residual.

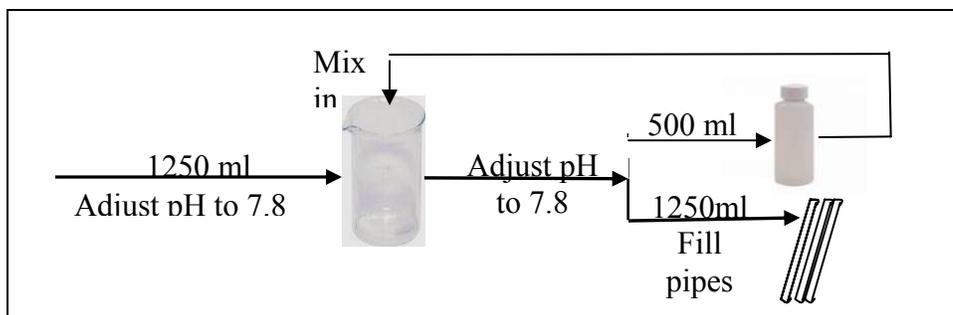


Figure 2. Water preparation for nitrification stimulation flow chart.

RESULTS AND DISCUSSIONS

Nitrification occurrence

During the first 11 months of the study when no mixture of old water was applied, nitrification was never detected via measurements of ammonia consumption and nitrite/nitrate production. Three months after starting the procedure of allowing old water to mix with new water; nitrification was detected in water with free ammonia. This nitrification was confirmed to come from nitrifiers present in Blacksburg tap water that survived breakpoint chlorination and became established in the plastic container in which the 500 mL water was allowed to sit stagnant until the next water change. The 1 mg/L $\text{NH}_3\text{-N}$ initially present in the water of this container was completely converted to nitrite within 24 hours, and the pH was decreased to below 7 after 24 hours.

Nitrification and pH changes in pipes

For waters containing ammonia, nitrification occurred in all pipes, though PVC pipes had much higher nitrification activity than lead, solder and brass containing rigs (Table 1). This is likely at least partly due to the toxicity of copper tube to nitrifiers. Nitrification was confirmed in these pipes by both ammonia consumption (Table 1) and correlated nitrite/nitrate increase. In addition, ammonia consumption and pH were found to have a good linear relationship in PVC pipes ($R^2 = 0.91$), indicating nitrification was the major cause of pH drop (Figure 3). However, in lead, solder and brass containing rigs, many factors can contribute to pH change in the rigs besides nitrification activity, including galvanic corrosion reactions and metal dissolution. Nitrification only contributed to part of the pH change, as the linear relationships between ammonia consumption and pH in these pipes had much smaller R^2 value than in PVC pipes ($R^2 = 0.23, 0.32, 0.52$ in lead, solder and brass containing rigs, respectively). Among these pipes, the pH in PVC and lead containing rigs dropped significantly (Table 1).

For waters containing monochloramine, ammonia consumption was also observed in every pipe (Table 1), but this ammonia consumption did not co-occur with increased nitrite/nitrate. In this event nitrification occurrence is deemed uncertain, and the extent of ammonia consumption is viewed as the upper bound to actual nitrification.

Table 1. Average ammonia consumption % and final pH after 24 hours stagnation in the indicated pipe material. Initial pH = 7.8.

	PVC	lead	solder	brass
	ammonia consumption %			
ammonia	97.0	29.9	24.2	32.9
monochloramine	16.8	26.1	23.3	19.2
	Final pH after 24 hour stagnation			
ammonia	7.24	7.39	7.51	7.85
monochloramine	7.58	7.68	7.60	7.92
chlorine	7.55	7.63	7.68	7.75
no-dis	7.79	7.56	7.56	7.93

Effect on Lead Release

A relative ratio of lead leaching was calculated to compare lead release difference among the four disinfectant types, in which the lead concentration determined for either monochloramine, chlorine or no disinfectant was divided by the level of lead from the pipe with ammonia:

Ratio =

$$\frac{\text{Pb (ppb) concentration with chloramine (chlorine, or no disinfectant) as disinfectant}}{\text{Pb (ppb) concentration with ammonia as disinfectant}}$$

If this ratio < 1 , lead leaching with ammonia is higher than with other disinfectants, whereas a ratio > 1 indicates that lead leaching with ammonia is lower than with other disinfectants. This result could indirectly indicate the effect of nitrification, since the pH drop in water with ammonia were partly caused by nitrification.

Before nitrification occurred in the pipe rig, for a given type of lead bearing material, differences in lead release due to different disinfectants were rarely significant at $> 90\%$ confidence. However, after nitrification began to occur in lead containing rigs, the water with ammonia had

the highest lead leaching (Figure 4). For solder and brass containing rigs, lead leaching was observed the same or lower with ammonia as with the other disinfectants (Figure 4). These results were consistent with the expectation that lower pH results higher lead leaching, since only lead containing rigs had lower pH when ammonia was used (Table 1).

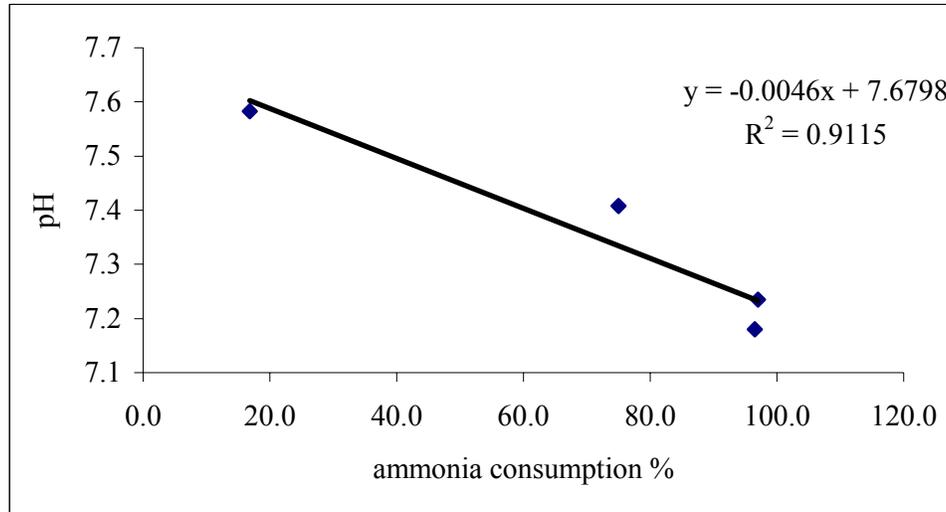


Figure 3. Drop in pH and ammonia consumption in PVC pipes after 24 hours stagnation.
(Data above includes data from other part of the test, which is not presented in this paper)

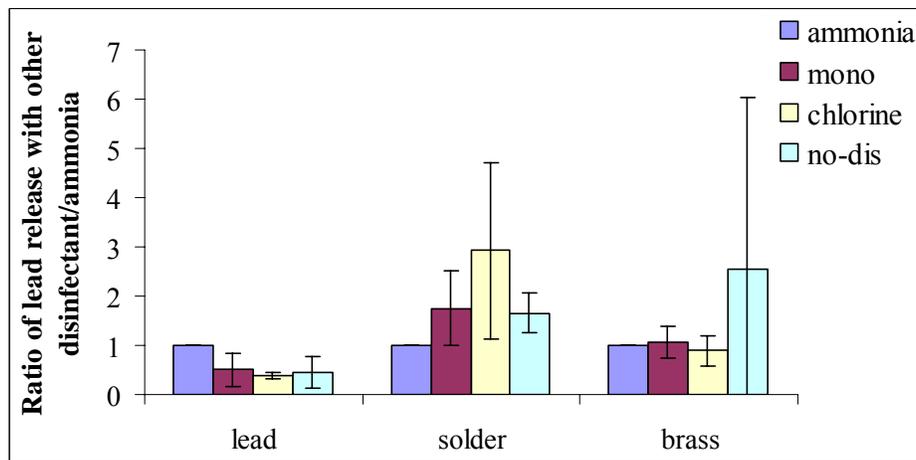


Figure 4. Lead release from different pipe configurations.
Error bars represent 90% confidence interval.

Effect on copper release

Before nitrification occurrence, ammonia was observed to have the lowest copper release among all disinfectants in lead containing rigs; but after nitrification occurred, copper release with ammonia was increased to similar to the other disinfectants (Figure 5). Again, this effect was attributed to pH drop in lead containing rigs, which was partly caused by nitrification. Copper release trends in solder containing rigs did not change after nitrification occurrence, as indicated by similar copper levels among the four disinfectant types. Copper release trends in brass containing rigs was very different before and after nitrification occurrence: before nitrification

occurrence, monochloramine had the highest copper release among the four disinfectant types, but after nitrification occurrence, copper release with no disinfectant and ammonia increased but copper release stayed at the same level with chlorine and no disinfectant. Possible explanations for the change in brass containing rigs might be that in pipes with ammonia and no disinfectant, biofilm on the pipes accumulated over time and had a negative effect on copper release. But for pipes with chlorine and monochloramine disinfectants, biofilm accumulation was controlled.

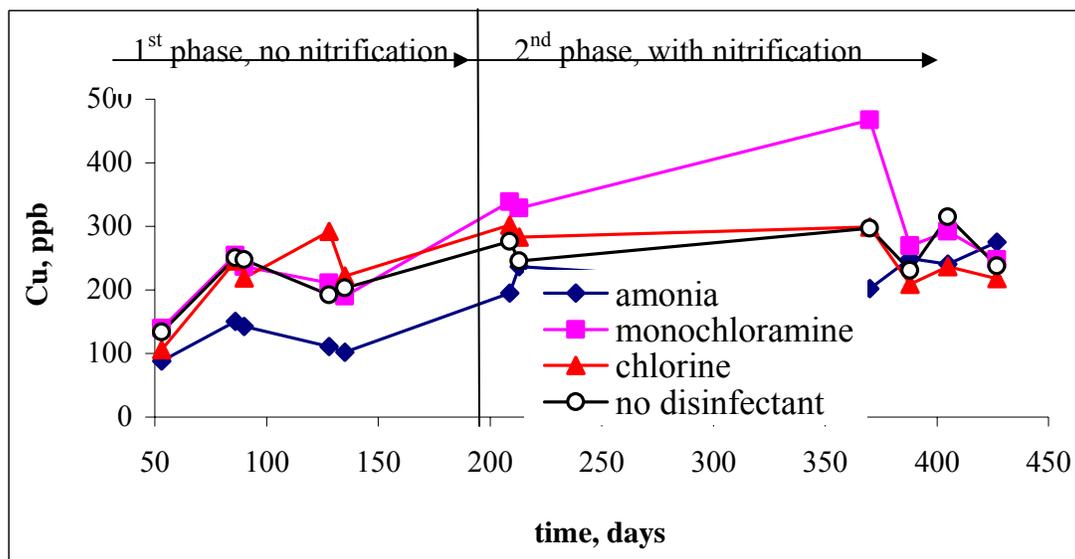


Figure 5. Copper release before and after nitrification occurrence in lead containing rigs.

CONCLUSIONS

Nitrification occurred in ammoniated water after mixing of old stagnant water. pH decreases in PVC and metal pipes were correlated to some extent with nitrification occurrence. The pH of water in lead containing rigs was significantly reduced, and this pH reduction caused increased lead and copper leaching.

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CORROSION IN HOME DRINKING WATER INFRASTRUCTURE: ASSESSMENT OF CAUSAL FACTORS AND COSTS

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KEY WORDS: pinhole leaks, corrosion, causes, costs

ABSTRACT

High incidences of pinhole leaks, which occur in home plumbing due to pitting corrosion of water pipes, have been observed in parts of the U.S. such as the Maryland suburbs of Washington D.C. This research evaluates factors associated with pinhole leak occurrences and assesses the costs incurred by consumers from corrosion. Statistical analysis of Maryland survey responses suggests that the probability of pinhole leak occurrences is associated with the type of pipes installed and the distance of the dwelling from a water treatment plant. The number of leaks and location of pinhole leaks in the dwelling drive the financial costs of pinhole leak damage. Research findings will inform policymakers, program managers, and water utilities on the importance of reducing corrosion in home drinking water infrastructure.

INTRODUCTION

Pinhole leaks are small holes in drinking water plumbing caused by pitting corrosion, a type of corrosion concentrated on a very small area of the inner pipe. Theories as to the origins of pinhole leaks vary and there are a few proven causes (Edwards *et al.* 2004). Edwards suggests that removal of natural organic matter mandated by tighter Environmental Protection Agency (EPA) drinking water standards can contribute to the problem in combination with other factors, since natural organic matter can be an inhibitor to the corrosion-inducing chemical reactions. Pipe failures can also result from other factors including faulty installation or wearing of copper due to friction by rubbing against a surface (Fleishman 2001).

In most cases, very small pinhole leaks are hard to detect especially if they appear in pipes running through walls and ceilings. Generally, it is recommended that pipes be replaced after three or four leaks (Gurner 2003), although others have recommended replacement after two leaks (Edwards *et al.* 2004). Damage from pinhole leaks can include collapse of walls and ceilings and the water can contribute to growth of mold on the surface of walls, floors, and ceilings. Mold exposure can cause allergic reactions, such as irritation of eyes, skin, and throat. Furthermore, copper itself can cause severe health problems such as liver and kidney failure, if consumed in doses higher than 1.3 mg/l (EPA 1992).

Pinhole leaks appear to be a nationwide problem, but they are more common in certain regions of selected states including Maryland, California, Florida, and Ohio. Some areas (for example, parts of Florida) have considered banning the use of copper piping in order to control the rising number of incidents (Gurner 2003). However, due to lack of information on pinhole leak and corrosion problems, building regulators and codes have been slow to address the issue in some areas.

The objectives of this research are 1) to evaluate the frequency and potential causal factors associated with pinhole leaks in home plumbing; and 2) to assess the financial and time costs incurred by consumers and their home insurers in repairing damages to plumbing and property resulting from pinhole leaks.

MARYLAND DRINKING WATER ASSESSMENT SURVEY

In July 2004, the Maryland Home Drinking Water Assessment Survey was conducted to learn more about the extent of pinhole leak problems in household drinking water plumbing systems. Mail surveys were sent to Maryland residents in the suburbs of Washington, D.C. to investigate their experiences with pinhole leaks. This area was selected because of the large number of pinhole leaks reported by utility customers to the Washington Suburban Sanitary Commission (WSSC). The survey sample was divided by zip code and zip codes with high numbers of reported pinhole leaks were sampled more heavily. A minimum of 10 surveys was sent to every zip code reporting leaks.

A total of 5,009 Maryland residents received the survey and 1,128 responses were returned of which 1,120 responses were used in the analysis. Eight responses were dropped from the study because they were incomplete. The survey analysis focused on the incidence of pinhole leaks; associated financial and time costs as well as potential causal factors. Samples were weighted to correct for oversampling in zip codes with high numbers of reported pinhole leaks. Responses were weighted by the number of people over 18 represented by each survey sample in each zip code.

CAUSAL FACTORS ASSOCIATED WITH PINHOLE LEAK OCCURRENCES

A probabilistic analysis of potential causal factors using a weighted logistic model was performed to predict the probability of pinhole leaks occurrences. To evaluate the logistic regression, the explanatory variables are related to the probability of pinhole leak incidents. The dependent variable is binary taking a value of 1 when pinhole leaks occur and 0 otherwise. The

following causal factors were employed in the model: geographical location of dwelling relative to water treatment plant (approximated by estimated water travel time to zip code in which the dwelling is located)¹, type of pipes installed (copper vs. others), history of pipe failure, time of pipe replacement (1960-1980), age of the dwelling, and the source of water (approximated by a dummy variable with a value of 1 for respondents located in zip codes which predominantly receive their water from Patuxent water treatment plant and 0 otherwise).² History of pipe failure reflects whether the respondent had other types of failure in their drinking water pipes beside pinhole leaks. Time of pipe replacement is a dummy variable with a value of 1 for respondents who indicated they had replaced or installed new drinking water pipes between 1960 and 1980. Age of the dwelling variable is approximated by a dummy variable with value of 1 for dwellings built before 1970 and value of 0 otherwise.

The unweighted number of observations with missing values is 241. The total number of unweighted observations with complete information on all variables utilized in the regression estimation is 879 (423 respondents with pinhole leaks and 456 respondents without pinhole leaks). Respondents without pinhole leaks were the control group in the logistic regression analysis. The logistic model is significant at a 5 percent level with Log Likelihood equal to 1,170 (Table 1).

The logistic regression results are presented in the form of probability values (Table 1). The estimated probability coefficients (Table 1) can be interpreted as the impact of one unit increase (decrease) in the independent variable on the chance of pinhole leak occurrences, while controlling for other variables in the model (UCLA Academic Technology Services, 2005). For example, one additional day of water travel decreases the probability of pinhole leaks by 37 percent. This finding is in agreement with Rushing and Edwards (2004) who found houses located closer to the treatment plant to be exposed to higher levels of chlorine and, therefore, to experience higher levels of pipe corrosion. Furthermore, copper pipes installed in the dwelling raise the chance of pinhole leak incidents by 65 percent; and history of pipe failures increases the probability of pinhole leaks by 69 percent. Pipe replacement, age of dwelling, and the source of water are not statistically significant.

Table 1: Effects of Selected Variables on Probability of Pinhole Leak Occurrences^a

Variables	Probability	P-value
Water travel time	-36.94	0.00 ^b
Copper pipes	64.85	0.00 ^b
History of pipe failure	69.23	0.00 ^b
Pipe replaced between 1960 – 1980	-48.35	0.86
Dwelling built before 1970	52.46	0.47
Source of water (Patuxent treatment plant)	51.40	0.74

^a Dependent and independent variables were weighted by sample weights. Base model: Log Likelihood = 1170; Pseudo R² = 0.053; n (unweighted) = 879.
^b Significant at 5%.

1 Estimated water travel times were provided by Robert Buglass, Principal Environmental Engineer, Washington Suburban Sanitary Commission, Laurel, Maryland. March, 2006.

2 Almost all respondents received water from WSSC.

COSTS ASSOCIATED WITH PINHOLE LEAK REPAIR

To assess the factors driving costs of pinhole leak repairs and associated damage, a weighted ordinary least squares (OLS) regression was estimated. The dependent cost variable includes the financial and time costs of pinhole leaks. Time spent dealing with pipe failures was translated into financial terms by multiplying the estimated time spent of the survey respondent by the estimated hourly wage rate for inhabitants in each zip code. The estimated hourly wage rate was calculated by employing annual average income for each zip code as reported by Census Bureau divided by the assumed number of hours worked (52 weeks*40 hours) in a year (Census Bureau, 2000). The estimated total expenses (financial and time costs) ranged from \$1,271 to \$18,455. Fifty-six observations had missing values for either the time or money spent on pinhole leak repairs. As a result, the total number of unweighted observations for total costs is 367. Total expenses were regressed on number of leaks, type of pipe material (copper vs. others; iron vs. others), type of water conveyed in the leaking pipe (hot vs. cold), place of leak in the dwelling, age of the dwelling, and source of water (Patuxent water treatment plant vs. other). Standardized coefficients of each variable were reported for the weighted ordinary least squares regression. They explain the impact of a one standard deviation increase in the independent variable on the dependent variable relative to other variables in the model. As a result, factor standardization allows for observing the relative importance of explanatory factors, while keeping the variable of interest in its original form (Kim and Feree 1981).

The weighted OLS estimation was performed on the observations that reported pinhole leaks. The number of leaks had a large positive impact on pinhole leak costs (Table 2). Copper plumbing and pipe failure on the first and second floor of a dwelling increased repair expenses. Plumbing system malfunctions under the slab and in the basement, on the other hand, were negatively related to the total repair expenses, probably because these areas were easier to access for repair and/or because leaks in these areas represented less associated damage to the structure and furnishings of the dwelling. According to a report by NAHB (1992), pinhole leak occurrences under slab and underground (16 unweighted observations) might be affected by abnormally aggressive soil with a high capacity of obtaining moisture. Iron pipes, cold water pipes, dwelling age, and source of water were not related to pinhole leak damage costs.

CONCLUSIONS

A survey investigating the pinhole leaks problem in the Maryland suburbs of the Washington, D.C. area was conducted in 2004. Analysis of survey results indicates that the probability of pinhole leaks depends on several factors. For example, the leak occurrences increase with use of copper pipes as the plumbing material and as the dwelling is located closer to the treatment plant. Leaks are also more likely to occur if a plumbing system has failed in the past. Pinhole leaks can have high financial and time costs. Repair costs increase with the number of leaks, repairs of copper pipes, and location of leaks on the first or second floor of the dwelling.

Due to the increasing frequency of pinhole leak occurrences across the nation, the Virginia Tech researchers working on this NSF-funded project have extended the investigation of pinhole leaks nationwide. Further research is being done on households' preferences for plumbing materials and their willingness to pay for improved plumbing infrastructure. The research findings will

inform policymakers, program managers, and water utilities on the importance of reducing corrosion in home drinking water infrastructure as well as inform consumers on the longevity and reliability of their plumbing systems.

Table 2. Total Costs of Repairing Pinhole Leaks and Associated Damage (Least Squares Regression)^a

Variables	OLS regression ^b	
	Standardized Coefficients	t-Statistic
Constant		-1.6
Number of leaks	0.26	5.1 ^c
Copper pipes	0.06	1.7 ^d
Iron pipes	0.02	0.3
Cold water pipes	-0.02	-0.4
Leak under slab and in basement	-0.19	-3.0 ^c
Leak on the first floor	0.29	5.2 ^c
Leak on the second floor	0.19	3.9 ^c
House built before 1960	-0.04	-0.9
Source of water	-0.06	-1.3
Selection bias correction	N/A	N/A

^a Observations weighted by sample weights.
^b $R^2 = 0.231$; F-stat = 12.48; n (unweighted) = 367.
^c Significant at 5%.
^d Significant at 10%.

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NATIONAL COSTS OF PINHOLE LEAKS IN DRINKING WATER INFRASTRUCTURE: PRELIMINARY ESTIMATES

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ABSTRACT

Pinhole leaks in drinking water plumbing are an increasing concern in some areas of the U.S. Surveys of homeowners, plumbing firms, and water utilities are used to estimate pinhole leaks rates and costs in the U.S. The total cost of pinhole leaks and pinhole leak prevention in the United States is estimated at \$928 million annually. The largest proportion of cost, \$534 million, accrues to owners of single-family homes, while multi-family apartment dwellings and commercial/public buildings incur \$87 million, and \$85 million, respectively. Approximately half of cost to single-family homes is for plumbing repairs, one third is time costs, and the remainder is for property damage. The cost to utilities of corrosion inhibitors and monitoring pinhole leak reports is estimated at \$222.4 million annually. Total costs may be higher than shown here because of expenditures of in-house plumbing repair services, defensive replumbing by risk averse homeowners, loss of treated water prior to use, and service line repair costs.

INTRODUCTION

Pinhole leaks in drinking water plumbing are an increasing concern in some areas of the U.S. Pinhole leaks can result in damage to homes and other buildings. Damage costs include pipe repair or replacement, costs of repairing damages to walls, floors, and furniture, and lost time to

the building owner. While concern has focused on corrosion and pinhole leaks in copper pipes, consumers also report pinhole leaks in other plumbing materials (Kleczyk *et al.* 2005). This paper provides national estimates of the costs of leaks in drinking water plumbing to residential and commercial property owners as well as costs of prevention by water utilities. This information will assist in assessing the need for corrective action to reduce potential damages from leaks in drinking water plumbing.

COST ESTIMATION PROCEDURES

National costs for pinhole leak damages and prevention are estimated for single-family detached homes, apartment/multi-family homes, commercial buildings, and utilities. The first 3 sectors incur costs from pinhole leak damages while utilities sector incurs costs of administering corrosion inhibitors to prevent corrosion. Costs are estimated for four ‘hotspot’ regions located in the Northeast, Southeast, Midwest, and West. Costs are also estimated for the rest of the U.S. The hotspot regions are referred to simply by name of the region in which they are located. The total cost for pinhole leaks in the U.S. is the sum of the costs by regions across sectors:

$$\text{Total annual pinhole leak cost} = \left[\sum_{s=1}^3 \sum_{r=1}^5 (\text{weight}_{sr}) * (\text{average cost per leak})_{sr} \right] + \text{utility pinhole leak costs}$$

The *s* sectors include single family (detached) homes, multi-family homes, and commercial buildings. *Weight_{sr}*, which reflects the proportion of costs incurred by each region and sector relative to the U.S. national cost, is based on the number of detached homes and the estimated annual leak rate in each region. Annual leak rates (estimated proportion of homes which have a pinhole leak each year) were calculated from a telephone survey of 780 homeowners conducted by Virginia Tech Center for Survey Research (Willis-Walton 2006a) and a mail survey of homeowners (1,120 responses) (Kleczyk *et al.* 2005). In each sample area, the average number of leaks per year per single-family dwelling was multiplied by number of single-family dwellings in 2004 (U.S. Census Bureau 2004). The estimated total number of single-family leaks, 938,231, is the sum of estimated leaks in all sample areas (Table 1). The calculation of costs per pinhole leak by sector is described below.

Table 1. Single-family detached homes and estimated annual leaks by hotspot.*

	Northeast	Southeast	Midwest	West	Rest of U.S.
Total number of detached homes (2004)	341,451	14,448	95,514	7,364,386	67,241,403
Annual leak rate all homes	0.074	0.047	0.015	0.016	0.012
Weight	25,276	682	1,401	119,854	791,018

* Hotspots are referred to by the name of the region in which they are located. The number of homes and leak rates refer to the hotspot.

Single-family Homes

Costs of pinhole leaks in detached homes are the estimated sum of homeowner time costs, repair costs and property damages paid by the homeowner and/or insurance company. Homeowners’ time costs are the value of time spent on attending to repairs and fixing damages caused by leaks which could otherwise be spent at work earning wages or other activities. Costs were inflated to

2005 dollars. Average cost per leak for each single-family home observation was estimated by dividing total costs of leaks by the number of leaks. Average cost per leak in each region is the sum of average costs per leak in all households with leaks divided by the number of observations in the region with leaks. Average costs were extrapolated to each region by multiplying average cost per leak in each region by the region's weight (Table 1).

Multi-family Residential and Commercial/Public Buildings

The costs of pinhole leaks in multi-family residential and commercial/public buildings were estimated by multiplying total single-family residential costs by a ratio, which reflects the proportion of multi-family to single-family residential costs or commercial/public building costs to single-family residential costs. Ratios are based on a national survey of 880 plumbing firms conducted by Virginia Tech Center for Survey Research (Willis-Walton 2006b) in which respondents were asked how much revenue their firms received from pinhole leak repairs in single-family residences, multi-family residential buildings, and commercial/public buildings, respectively. The ratio of revenue from multi-family residential to single-family residential pinhole leak repairs or commercial/public building pinhole leak revenue to single-family pinhole leak revenue was calculated for each respondent. The averages of the estimated ratios for plumbing firm respondents in each region are used. For example, if 73%, 22%, and 5% of pinhole leak repair income came from single-family residences, multi-family residences, and commercial/public buildings, respectively, the multi-family to single-family cost ratio is $22/73 = 0.3$, and the commercial building to single-family cost ratio is $5/73 = 0.07$.

Utility Systems

Utility system costs included costs of adding corrosion inhibitors to treated water and costs of monitoring customer reports of pinhole leaks. Costs were estimated based on a national mail survey of water utilities (133 responses) (Bosch *et al.* 2006) and a followup survey (21 responses). Corrosion inhibitor costs include cost of purchasing inhibitors, capital and labor costs of adding inhibitors to water, and cost of removing inhibitors as part of waste water treatment. Economies of size in treating water with inhibitors (due to greater efficiencies in applying corrosion inhibitors and spreading fixed costs of applying inhibitors over more water) are accounted for by estimating costs by four utility size categories: small (less than 10,000 connections), medium (between 10,000 and 50,000 connections), large (between 50,000 and 100,000 connections), and very large (more than 100,000 connections).

Utilities' capital costs include depreciation and interest expenses for capital equipment for applying inhibitors including buildings and containment structures, chemical tanks, and feed pumps. Annual depreciation and interest were calculated by amortizing the total value of the buildings and equipment over their useful lives assuming an interest rate of 6.5%. Respondents estimated the construction or purchase cost and useful life of equipment and buildings. Costs of removing inhibitors from wastewater are estimated to be equal to that of the cost of purchasing inhibitors based on the experience of one utility (Buglass 2006). Pinhole leak monitoring costs were estimated based on reports from eight utilities who provided estimates of staff time required for monitoring pinhole leak reports per year and the hourly cost of staff time.

Volume of water treated with inhibitors by utilities in each size class was estimated based on the national utilities survey. Most utilities in every size class used inhibitors; however the

proportion is higher in the larger utilities size class. The percentage of utilities using inhibitors in each size class was multiplied by estimated water treatment of utilities in that size class to obtain the total amount of water treated with inhibitors. The AWWA (2006) estimates that the U.S. processes approximately 34 billion gallons of water per day or 12.41 trillion gallons of water annually. Very large, large, medium, and small utility respondents treat 48, 30, 19, and 3 percent of this total, respectively, based on responses to the national water utility survey. These percentages were multiplied by total water production to obtain estimated water production by size class of utilities.

RESULTS

Single-family, multi-family residential, and commercial/public buildings

Hotspot regions tended to have higher costs per leak compared with the rest of the U.S. Average costs per leak in hotspot regions varied from \$579 in Midwest to \$1,698 in Southeast compared to \$456 in the rest of the U.S. Property damages accounted for a large share of higher costs in hotspot regions. Property damage costs per leak varied from \$372 to \$782 in hotspot regions (46 to 63% of total costs) compared to \$39 (9% of total costs) in the rest of the U.S.

Total annualized costs of pinhole leaks in single family homes are \$533 million (Table 2). Hot spot regions account for \$173 million (32%) compared to \$361 million (68%) in the rest of the U.S. Repair cost comprises the largest proportion of overall costs (46%) followed by time costs (32%) and property damage costs (22%). Of the \$534 million total cost in single-family homes, \$308 million (58%) is allocated to copper materials. Plastic/PVC and ‘other materials’ account for almost equal shares, \$113 and \$112 million, respectively. While corrosion occurs in metal only, some homeowners surveyed reported pinhole leaks occurring in plastic pipe. The leaks reported in plastic may have been due to another type of material failure other than corrosion.³

Total cost of pinhole leaks in all types of buildings is \$706 million of which \$222 million (31%) is in hotspot regions, and \$484 million (69%) is in the rest of the U.S. (Table 3). Seventy-six percent of total cost (\$534 million) is in single-family homes. The remainder is divided almost equally between multi-family residential buildings (\$87 million) and commercial/public buildings (\$85 million). The percent breakdown of costs among building types varies among regions with single-family homes comprising 73 to 78% of total costs. Multi-family buildings comprise 11 to 22% of costs and commercial/public buildings comprise 5 to 14%.

Cost estimates for multi-family and commercial/public buildings may be conservative because managers of many multi-family and commercial/public buildings may have in-house personnel who perform plumbing maintenance and repairs. Pinhole leak repair costs for these situations would not be captured based on respondents to the plumbers survey, who represented commercial plumbing operations. Responses to the mail survey from residents in multi-family dwellings were used to estimate leak rates in multi-family dwellings directly. These direct estimates resulted in leak rates more than double the rates obtained using the indirect method

³ Some respondents may have identified other types of leaks or plumbing failures as pinhole leaks. Respondents were given the following information about pinhole leaks. “A pinhole leak is a small leak located directly on a water pipe, and may often be seen as a steady drip. A pinhole leak is not a dripping faucet or leaking toilet.”

based on plumbers' reports. If this trend holds nationally, costs in multi-family and commercial/public buildings could be double those shown in Table 3.

Table 2. Total annualized costs of pinhole leaks in single-family homes in hotspots and rest of the U.S.*

	Northeast	South-east	Midwest	West	Rest of U.S.	Total
	Thousands of dollars					
Repair cost	17,673	516	210	39,971	187,724	246,098
Percent**	84	45	26	27	52	46
Property damage cost	included in repair costs	533	521	84,766	31,142	116,961
Percent**		46	63	57	9	22
Time cost	3,453	108	80	25,251	141,869	170,780
Percent**	16	9	12	17	39	32
Total cost	21,127	1,158	812	149,988	360,735	533,840
*Hotspots are referred to by name of region in which they are located.						
**Percent of total cost for indicated hotspot or rest of the U.S.						

Table 3. Total pinhole leak costs in single- and multi-family residential and commercial/public buildings.

	Northeast	South-east	Mid-west	West	Rest of U.S.	Total
	Thousands of dollars					
Single family home	21,127	1,158	812	149,988	360,735	533,840
Percent	76	73	77	78	75	76
Multi-family bldg.	4,830	346	165	28,185	53,523	87,048
Percent	17	22	16	15	11	12
Comm./publ. bldg	1,856	84	82	13,394	69,384	84,801
Percent	7	5	8	7	14	12
Total costs all bldgs.	27,813	1,587	1,059	191,566	483,642	705,689

Utility costs

The national cost of adding corrosion inhibitors to drinking water and removing them from wastewater is an estimated \$222 million annually (Table 4). The costs of purchasing and removing inhibitors are the major costs, \$90 million each, followed by capital costs and labor costs in that order. Medium size utilities have the largest inhibitor costs due to high cost per unit

of water treated. Very large utilities have the next highest cost due to the large amount of water they treat with inhibitors. Annual monitoring costs of monitoring pinhole leak complaints are \$350,174, which increases total utility inhibitor and monitoring costs to \$222.4 million.

SUMMARY AND CONCLUSIONS

The total cost of pinhole leaks and pinhole leak prevention in the United States is estimated at \$928 million annually. The largest proportion of cost, \$534 million, accrues to owners of single-family homes, while multi-family apartment dwellings and commercial/public buildings incur \$87 million, and \$85 million, respectively. Approximately half of cost to single-family homes is for plumbing repairs, while one third is time costs, and the remainder is for property damage. The cost to utilities for adding corrosion inhibitors and tracking pinhole leak reports by customers is estimated at \$222.4 million annually.

Several factors not quantified in the analysis may cause total costs to be higher than shown here. First, the method does not account for commercial and apartment buildings with in-house plumbers who perform repairs on pinhole leaks. Second, the estimates do not account for risk averse (Robison and Barry 1987) homeowners who may replumb before any leaks are evident in order to avoid large damages. Third, pinhole leaks result in the loss of treated water prior to use by consumers. Fourth, the cost estimates do not include costs to utilities to repair service lines with pinhole leaks. Utilities' costs of preventing pinhole leak corrosion should perhaps be lower than estimated here, because utilities add inhibitors for multiple reasons some of which are not related to preventing pinhole leaks.

Table 4. Annual water utility costs of adding and removing corrosion inhibitors.

Utility size—total connections	Purchase cost	Removal cost	Labor cost	Capital cost	Total cost
Thousands of dollars					
Very large >100,000	29,577	29,577	549	4,306	64,009
Large 50,000 - 100,000	25,315	25,315	1,157	7,690	59,478
Medium 10,000 - 50,000	29,972	29,972	5,663	11,829	77,437
Small < 10,000	5,161	5,161	9,241	1,212	20,774
Total	90,025	90,025	16,610	25,037	222,048

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A CASE STUDY FOR COMPARATIVE PERFORMANCE ASSESSMENT AND EVALUATION OF CONSOLIDATING CWS

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ABSTRACT

Decreasing water resource availability, stricter water quality regulations, decreasing federal subsidy levels, increasing public scrutiny, decreasing financial health, and increasing infrastructure replacement costs are forcing community water systems (CWS) to change the way they operate. The combination of these forces has led to consolidation among CWS that operate as a regional water system (RWS). Consolidating CWS require the use of uniform metrics that permit assessment of performance against consistent criteria, and evaluation of the results against a standard that is fair and applicable to all the decision-making units (DMU) of the RWS. Rogers (2005) developed the theoretical basis for a method for comparative performance assessment and evaluation (CPAE). A case study was completed in 2004 to validate the efficacy of the proposed CPAE method. The case study involved comparative performance analysis of ten CWS, within the service area of Region 2000, in south central Virginia. The conclusion of the case study is that by using the proposed CPAE method, consolidating CWS can collectively save approximately 11% by efficiently allocating limited financial resources among similar DMU of the RWS.

INTRODUCTION

Decreasing water resource availability, stricter water quality regulations, decreasing federal subsidy levels, increasing public scrutiny, decreasing financial health, and increasing infrastructure replacement costs are forcing community water systems (CWS) to change the way they operate (Maxwell, 2005; Standish-Lee *et al.* 2006). The combination of these forces necessitates consolidation responses among CWS to operate as a regional water system (RWS), especially within the same geographic region, *i.e.*, watershed (Shih *et al.* 2004, Young 2006). This regional consolidation response by CWS is intended to yield economies of scale by eliminating non-structural redundancies in engineering, administrative, and institutional functions (Shih *et al.* 1994).

However, the consolidation response by CWS creates a need for integrated planning and management approaches that facilitate new transparent and collaborative decision-making processes to maintain and compare levels of service (Heller *et al.* 1999). This paper proposes

one solution for consolidating CWS to maintain and compare levels of service in an environment of limited financial resources is to implement a method for comparative performance assessment and evaluation (CPAE). The purpose of CPAE is to develop and utilize uniform metrics that permit assessment of performance against consistent criteria, and evaluation of the results against a standard that is fair and applicable to all of the decision-making units (DMU) of the RWS (Westerhoff *et al.* 1998, Andrews *et al.* 1999).

The proposed CPAE method provides a transparent and consistent decision-making approach for assessing the performance and evaluating the assessment results of consolidating CWS (Rogers and Louis 2005). The outcomes of the proposed CPAE method also facilitate improved accountability of the individual community water system provider (CWSP) to its' stakeholders. Rogers (2005) developed the theoretical basis for such a CPAE method. A case study was completed in 2004 to validate the efficacy of the proposed CPAE method. The case study involved comparative performance analysis of ten CWS, within the service area of Region 2000, in south central Virginia. The goal of the case study was to improve the accountability of consolidating CWS to their stakeholders. The objectives of the case study were to demonstrate the value of the proposed CPAE method by providing a: (1) Summative performance evaluation method that permits benchmarked comparison; and (2) Decision Support System (DSS) tool that facilitates the efficient allocation of limited financial resources among consolidating CWS. The six steps of the proposed CPAE method for consolidating CWS that are operating as a RWS are detailed in the following section.

METHODS

1. Define the Product Life Cycle for Drinking Water

The universal goal for public services, such as CWS, is to maximize levels of service at the lowest possible cost, while maintaining quality objectives (Saad 1994; Akosa *et al.* 1995). This systems' goal requires the CWSP to maximize the percentage of the total raw water input volume that is consumed relative to the full cost of drinking water production (Rogers 2005). The generic production process of converting raw water into drinking water is defined through a product value-chain that includes raw water storage, raw water intake, raw water transmission, raw water treatment, finished water transmission, finished water storage, finished water consumption, and finished water disposal (Louis & Siriwardana 2003). This product value-chain for drinking water can be compared to the traditional life cycle stages for conception, development, production, utilization, reuse, and disposal (Ciambrone, 1997). This comparison leads to a definition for the product life-cycle of drinking water that reflects the full cost to acquire, produce, store, deliver, and dispose of an acceptable quantity of drinking water relative to the unit amount of financial resources utilized (Arditi and Messiha 1999).

2. Define the Acceptable Output or MG_{Net} Variable for the CWSP

The product life cycle of drinking water provides the basis for defining a standard efficiency metric (SEM) parameter for the CWSP (Rogers and Louis 2005). The traditional efficiency metric is defined as the ratio of the acceptable output divided by the input resources utilized. In the CWS sector, the traditional efficiency metric is defined as the quantity of drinking water that is produced per dollar utilized. However, this efficiency metric does not evaluate the full cost of drinking water over its' entire product life cycle. Therefore, a new output variable for the

proposed SEM parameter is defined over the product life cycle of drinking water, as illustrated by Equations 1 and 2:

$$MG_{FinishedWater} = MG_{RawWater} - MG_{ProcessLosses} \quad \text{Equation 1}$$

$$MG_{NetWater} = MG_{FinishedWater} - MG_{UnaccountedLosses} \quad \text{Equation 2}$$

Where:

$MG_{Raw\ Water}$ = quantity in million gallons per year taken into treatment processes

$MG_{Process\ Loss}$ = quantity in million gallons per year lost by treatment processes

$MG_{Finished\ Water}$ = quantity in million gallons per year leaving treatment processes

$MG_{Unaccounted\ Loss}$ = quantity in million gallons per year lost by infrastructure leakage

$MG_{Net\ Water}$ = quantity in million gallons per year metered or consumed

3. Define the Financial Resources Utilized or $\$_{SC}$ Variable for the CWSP

The ability of the CWSP to standardize its input resources utilized variable is governed by the implementation of Governmental Accounting Standards Board (GASB) Statement Number 34 entitled, *Basic Financial Statements and Management's Discussion and Analysis for State and Local Governments* in 2000. The GASB-34 requirements standardize the municipal financial reporting mechanisms and provide guidance regarding performance measurement. Donahue and Hellenbrand (2002) concluded that the implementation of the GASB-34 requirements is important because external financial reporting can demonstrate financial accountability to the public and is the basis for investment, credit, and many legislative and regulatory decisions. Therefore, the input or financial resources utilized variable for the proposed SEM parameter is defined as the amount of total annual service expenditures for the CWSP. Therefore, the amount of the financial resources utilized represents the full-cost to acquire, produce, store, and deliver a unit quantity of drinking water over its entire product life cycle.

4. Construct New SEM and 1/SEM Parameters for the CWSP

The data envelopment analysis (DEA) technique provides the theoretical foundation for the development of the proposed SEM parameter for the CWSP. Charnes *et al.* (1978), Banker & Morey (1986), Saad (1994), Sexton *et al.* (1994), Akosa *et al.* (1995), and Shih *et al.* (2004) utilized the DEA technique for measuring and comparing relative efficiencies among similar DMU, such as consolidating CWS operating as a RWS. The DEA technique provides a nonparametric approach to construct a relative efficiency ratio or SEM parameter between the $MG_{Net\ Water}$ and $\$_{SC}$ variables of the CWSP, as illustrated in Equation 3:

$$SEM = MG_{NetWater} \div \$_{SC} \quad \text{Equation 3}$$

The proposed SEM parameter provides a uniform and consistent measure of performance for calculating the level of service for the CWSP. The proposed SEM parameter determines how many gallons of drinking water are efficiently consumed per unit dollar expended. The CWSP with the highest SEM parameter value is the most efficient organization among similar DMU. The ability of the CWSP to efficiently utilize its financial resources is calculated by its' cost-

efficiency metric. The inverse of the proposed SEM parameter is defined as the standard cost-efficiency metric, as illustrated in Equation 4:

$$1/SEM = \$_{SC} \div MG_{NetWater} \quad \text{Equation 4}$$

The 1/SEM parameter determines how many dollars are expended per unit quantity of drinking water that is efficiently consumed. The CWSP with the lowest 1/SEM parameter value is the most cost-efficient organization among similar DMU.

5. Develop the New DEA-Based LP DSS Tool for the RWS

The above DEA technique does not have the ability to distinguish the efficient or optimal solution to a resource allocation problem.. However, the allocation of limited resources among similar DMU can be represented as a linear programming (LP) problem. Ignizio and Cavalier (1994) formulated a LP problem for resource allocation that optimizes an objective function relative to limiting production and resource constraints, as illustrated by Equation 5:

$$Z = \text{Max } f(x) \quad \text{Equation 5}$$

Where:

x is the factors, *i.e.*, control variables that are subject to change; and

Z is a mathematical function that represents the desires of the decision-maker. The magnitude of the objective function provides the efficient solution to the resource allocation problem.

s.t.

$f(x) \leq b$, b represents production and resource constraints on the control variables; and

$x \geq 0$, represents non-negativity constraints on the control variables

The objective of the LP resource allocation problem is to maximize or optimize the budgetary allocations among the similar DMU. This optimization requires tradeoffs among the production and financial resource constraints of the similar DMU. Finally, the adverse impacts of these constraint tradeoffs on the optimal solution is mitigated by the ability of the DMU to be efficient relative to its' peers. The inclusion of the impact of the DEA-based 1/SEM parameters within the proposed DSS tool is an extension of the LP resource allocation problem. The CWSP's 1/SEM parameter is incorporated into a LP resource allocation problem as a relative cost-efficiency constraint. Specifically, the magnitude of the CWSP's 1/SEM parameter must be less than or equal to a regional cost-efficiency target value.

6. Characterize the Financial Resource Allocation Decision for the RWS

The DEA technique and LP resource allocation algorithm provides the basic mathematical foundation for the proposed DEA-based LP DSS tool. The proposed DSS tool utilizes the DEA technique to develop simple ratio relationships between the CWSP's $MG_{NetWater}$ and $\$_{SC}$ variables to construct uniform 1/SEM parameters that evaluate the most efficient DMU among s CWS. Next, the proposed DSS tool utilizes the LP simplex optimization algorithm to efficiently allocate the limited RWS budgetary dollars among s CWS. The magnitude of the Z efficient RWS budgetary allocation solution is equal to the sum of the x_s efficient budgetary allocations among s CWS. The Z for the decision variables x_s is attained through optimizing tradeoffs

among the levels of the production, financial, and cost-efficiency constraints for s CWS as follows:

- a. $(MG_{Net\ Water})_s = \text{constant}$, where the level of the sth CWSP MG_{Net} variable is held constant throughout the analysis period;
- b. $x_s \leq (\$_{SC})_s$, where the level of the sth CWSP x_s parameter is allowed to take on values that are less than or equal to the level of its' $\$_{SC}$ variable; and

c. $x_s \div (MG_{NetWater})_s \leq \min \left[(1/SEM)_s, \left(\sum_s (\$_{SC})_s \div \sum_s (MG_{NetWater})_s \right) \right]$, where the

level of the sth CWSP 1/SEM parameter is allowed to take on its' value if less than or equal to the level of the regional cost-efficiency target for the RWS. Otherwise, it takes on the value of the regional cost-efficiency target.

The magnitude of the CWSP's $MG_{Net\ Water}$ variable is demand driven, which means that the decision-maker can indirectly influence this output variable through waste reduction, metering accuracy, and loss prevention programs. The magnitude of the CWSP's $\$_{SC}$ variable is management-driven, which means that the decision-maker can directly influence this input variable by deciding when and how to fund certain line item expenditures during a periodic budgetary adjustment process. The $Z = \sum_s (x)_s$ for the efficient x_s parameters of the CWS represents the efficient total budget for RWS. Since the CWSP's $MG_{Net\ Water}$ variables are held constant over the analysis period, the level of Z for RWS is controlled by changes to the levels of the CWSP's $\$_{SC}$ variables for s CWS. The changes in the magnitude to the CWSP's $\$_{SC}$ variables among s CWS are controlled by setting the magnitude of the regional cost-efficiency target for the RWS.

For this paper, the value of the regional cost-efficiency target is defined as the estimate of the magnitude of the "average 1/SEM parameter" for s CWS. Additional assumptions for the DEA-based LP DSS tool include: (1) CWS are considered integral subsystems of the RWS that seek to obtain a systems' goal to maximize level of service at the lowest possible cost, while maintaining associated quality objectives; (2) Consolidating CWS of the RWS represent similar DMU; (3) Quantity of the CWSP's $MG_{Net\ Water}$ variable is constant throughout the analysis period; (4) Amount of the CWSP's $\$_{SC}$ variable is controllable throughout the analysis period; (5) Degree of attainment of the RWS's goal is measured by the magnitude of the CWSP's 1/SEM parameter; (6) Generation of profits is not the driving force as the decision-maker of the RWS strives to balance revenues with expenses during the annual budgetary process; and (7) Rational decision-maker of RWS seeks to efficiently allocate limited financial resources among DMU.

RESULTS

A case study for CPAE was completed in 2004 to test the hypothesis that ten independently operating CWS, within the service area of Region 2000, in south central Virginia, could achieve budgetary savings by consolidating and operating as a RWS. The s=10 independent CWS now operate under the existing budgetary allocation strategy with $(\$_{SC})_s=10$ independent budgets. The proposed method for CPAE assumes that an efficient budgetary allocation strategy is attainable, whereby the s=10 similar DMU agree to consolidate and operate as a RWS with $Z=1$

efficient regional budget. The efficient RWS budgetary allocation solution Z will be broken down as $x_s = 10$ efficient CWSP budgetary allocations as determined by optimizing tradeoffs among the CWS' ($MG_{Net\ Water})_s$ and $(\$SC)_s$ constraints relative to attaining the RWS's average cost-efficiency or $\{[\sum_s (\$SC/MG_{Net\ Water})_s]/s\}$ constraint.

The case study for CPAE began by collecting and analyzing the annual output and input data from 1998 through 2002 for each CWSP. Then, this data were used to calculate estimates of the five-year average values for the CWS' $MG_{Net\ Water}$ variables, $\$SC$ variables, and $1/SEM$ parameters. Finally, the proposed DSS tool was used to calculate the efficient solution for allocating RWS budgetary resources among CWS, as shown in Tables 1, 2, and 3:

Table 1. Average Values for $MG_{Net\ Water}$ and $\$SC$ Variables for Ten CWS in Region 2000.

Data	S ₁	S ₂	S ₃	S ₄	S ₅	S ₆	S ₇	S ₈	S ₉	S ₁₀
MG_{Net}	73	128	460	616	40	617	391	314	80	3289
$\$SC$	170,581	361,520	1,382,350	1,005,818	165,496	1,155,712	1,988,739	655,848	760,704	8,327,637

Note: The annual output ($MG_{Net\ Water}$ – Million Gallons) & input ($\$SC$ – Dollars Expended) data for each CWSP are consolidated into estimates of their average values for the analysis period from 1998 through 2002 to smooth out the effects of the annualized data outliers on the magnitudes of the predicted output & input variables within the mathematical model.

Table 2. Average Values for SEM & $1/SEM$ Parameters for Ten CWS in Region 2000.

	S ₁	S ₂	S ₃	S ₄	S ₅	S ₆	S ₇	S ₈	S ₉	S ₁₀
$MG_{Net}/\$SC$	0.00043	0.00035	0.00033	0.00061	0.00024	0.00053	0.00020	0.00048	0.00011	0.00039
$\$SC/MG_{Net}$	2,337	2,824	3,005	1,633	4,137	1,873	5,086	2,089	9,509	2,532

$MG_{Net\ Water}$ – Million Gallons & $\$SC$ – Dollars Expended

Table 3. Results for the DEA-based LP DSS Tool for Ten CWS in Region 2000.

Total \$	S ₁	S ₂	S ₃	S ₄	S ₅	S ₆	S ₇	S ₈	S ₉	S ₁₀
\$15,974,405	170,581	361,520	1,382,350	1,005,818	165,496	1,155,712	1,988,739	655,848	760,704	8,327,637
\$14,238,105	170,601	340,352	1,223,140	1,005,928	106,360	1,155,641	1,039,669	655,946	212,720	8,327,748

Row 1 is the actual $\$SC$ variables under current independent CWSP budgetary allocation approach.

Row 2 is the efficient RWS budgetary allocation solution Z for x_s parameters from the DSS tool.

Table 3 results prove the case study hypothesis that if the ten CWS, within the service area of Region 2000, operate as a RWS they could save approximately 11% over the current practice of independent CWSP budgetary allocation approach. The DEA-based LP DSS tool displays the predicted amounts of the x_s parameters for each CWSP relative to its' ability to attain the level of the selected RWS cost-efficiency target, which is representative of the acceptable CWSP level of service within the RWS. A formal sensitivity analysis of the variables and parameters of the DEA-based LP DSS tool is easily accomplished by simply changing the magnitudes of the $MG_{Net\ Water}$ and $\$SC$ variables for a respective CWSP.

DISCUSSION

The case study demonstrates the value of the proposed CPAE method as a method for summative performance evaluation that permits the benchmarked comparison of consolidating CWS. There were significant variations in the magnitudes of the SEM and 1/SEM parameters under the existing service resource allocation approach for independent CWS because they did not uniformly and consistently measure, monitor, or report on the temporal and spatial changes to their levels of service.

The case study also demonstrates the value of the proposed DEA-based LP DSS tool for CPAE to efficiently allocate limited financial resources among consolidating CWS. The results of the case study show that the limited financial resources distributed under the existing service resource allocation approach for independent CWS are not efficiently allocated, as demonstrated by the 11% savings from the efficient financial resource allocation solution.

Finally, the proposed SEM and 1/SEM parameters of the DEA-based LP DSS tool provide excellent benchmarks regarding the level of service of the CWSP relative to its' peers. The proposed 1/SEM parameter also provides an excellent measure of performance to efficiently allocate limited financial resources among consolidating CWS. However, the key to the effectiveness of the 1/SEM parameter as an relative cost-efficiency constraint is the selection of the regional cost-efficiency target. The regional cost-efficiency target provides the stakeholders with an objective mechanism to aid in the efficient allocation of limited financial resources among consolidating CWS.

CONCLUSIONS

A combination of conflicting forces within the CWS sector requires the development of a method for the CPAE for consolidating CWS. Consolidating CWS require new integrated planning and management approaches that facilitate transparent and collaborative decision-making processes. The purpose of the proposed method of CPAE is to improve the accountability of consolidating CWS to their stakeholders.

The proposed method for CPAE is superior to the existing PAE methods within the CWS sector because it defines and utilizes uniform and consistent measures of performance that focus on the attainment of a systems' goal for similar DMU. Additionally, the proposed method for CPAE does not require the identification of a complex mathematical relationship between the output and input variables of the consolidating CWS. Finally, the proposed method for CPAE limits the magnitude of the adverse impact to underperforming CWS by only reducing the amount of its financial resource input to a point where the level of its cost-efficiency is less than or equal to a maximum regional cost-efficiency target.

The results of the case study for CWS , within the service area of Region 2000, in south central Virginia revealed that they do not: (1) Adequately monitor, track, and report on temporal and spatial changes in levels of service; and (2) Efficiently allocate limited financial resources under the current independent service scenario. The conclusion of the case study is that by using the

proposed method for CPAE, consolidating CWS can collectively save 11% by efficiently allocating limited financial resources among similar DMU.

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DRIP DISPERSAL SYSTEMS

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ABSTRACT

Drip dispersal systems for dispersal of secondary or better treated sewage effluent into soils have become wide spread in Virginia since the middle 1990's. Drip dispersal allows placement of the infiltrative surface much higher in the soil profile, or even at or above the existing soil surface thereby providing necessary hydraulic gradients for water to move away from the application point and be assimilated by the natural landscape and its flora and fauna.

Drip dispersal systems routinely require secondary treatment of sewage with some form of aerobic treatment to minimize clogging of both the drip dispersal system and soil pores. Drip dispersal with septic tank effluent is allowed in Virginia and some other States but is not the predominant choice. With secondary effluent, the soil serves primarily as a polish and to disperse the effluent (in contrast to a system with septic tank effluent where the soil would be the primary treatment mechanism). A smaller foot print is required for drip systems due to their efficiency in dispersing effluent evenly across the dispersal area. Due to their more complex design using active electrical and mechanical components, drip systems and the pretreatment component require routine inspection and maintenance.

INTRODUCTION

Today more than 25 million homes, representing about 25% of the homes in the United States, rely on some form of onsite wastewater treatment and dispersal system. Most of these are a septic tank with trenches for dispersal of effluent (Jantrania and Gross 2006). As population growth has occurred in the United States in areas not feasible for central sewage collection and treatment systems, soils traditionally considered suitable for on-site treatment and disposal systems have become more scarce. This has led to development of alternatives to septic tanks

with dispersal of effluent to gravel filled trenches or pits. This evolution is well illustrated by Jantrania and Gross (2006).

This discussion will be provided relative to Virginia Department of Health Sewage and Handling Regulations but the principles apply regardless of the geographical location.

REASONS FOR USING DRIP DISPERSAL SYSTEMS

Key reasons for using drip dispersal systems include:

- Presence of soils too shallow rock or groundwater to comply with regulations allowing septic tank effluent or secondary treated effluent to be dispersed via trenches.
- Presence of limited area and or step slopes along with the above conditions.
- Presence of a large flow from a decentralized system in which it is more economical to use drip dispersal than trench dispersal and easier to manage flows.

DESIGN FACTORS BASED ON SOILS

System Foot Print

Drip system loading rates in Virginia for single family homes are based on the concept of a foot print area. Whereas conventional and LPD trench dispersal systems are sized in square feet of trench bottom area as a function of estimated percolation rate, when the area between the trenches and the trenches are combined, this total area comprises the footprint of the dispersal system. The minimum area therefore for single family dwellings is considered to be three times the area prescribed for an LPD system for a soil with a given LPD system loading rate (Sewage and Handling Regulations, Table 5.4 Area Requirements for Absorption Trenches).

For systems with flows exceeding 1000 gpd which are classified as mass drain fields, policy has been developed which allows trench bottom loading rates up to 25% of measured saturated conductivity rates (GMP # 101 Large Waste Water Systems). While this is perhaps liberal, especially in the presence of nearby underlying restrictive layers, it does reflect the understanding that pretreatment of effluent dramatically reduces the potential for soil clogging. However care must be taken that the water can be expected to be transmitted away from the infiltration point, especially during wet conditions on flat sites with low subsoil permeability. Tyler and Converse (1990, 2000) have introduced the concept of linear loading rate when restrictive conditions are present to limit the potential for seepage from down slope areas of a system that is narrow across the slope but long down the slope.

As slope increases, the trench spacing increases to offset the difference between slope distance (the hypotenuse of a right triangle) and the horizontal distance (the base of the triangle). Likewise, the dispersal tubing spacing is required to increase as a function of slope and separation distance between the infiltration point and depth to rock, *etc.* The typical tubing spacing is two ft. with a typical orifice spacing of two ft.

Absorption Area Increase (percent)				
Depth to Impervious Strata	Slope of site (percent)			
	10-19	20-29	30-39	40-50
Drip Tubing <24" off rock, pans, or impervious strata	17%	33%	50%	67%
Drip Tubing 24" or more off rock, or impervious strata	0%	17%	33%	50%

(Table from GMP 107, Virginia Department of Health, Office of Environmental Health Services, August 2, 2000)

Installation Depth

Under current regulations, trench dispersal systems installed in a clay loam soil horizon, must have at least 30 to 56 inches total depth of well drained soil depth for slopes ranging from 0 to 50 percent. The depth requirement increases by one-half inch for each % slope.

Unlike trench dispersal systems, the depth of installation for drip dispersal systems is not required to increase as the slope increases. While a stand-off of 12 inches above rock or the water table and 18" to impervious strata is still required, drip tubing can be placed as shallow as the surface. This is perhaps the most significant advantage drip dispersal systems have in the Virginia Regulations which is especially beneficial in hilly, wooded and mountainous areas with shallow limiting depth to restrictive layers.

The optimal installation depth in temperate climates seems to be 6 - 12 inches. Virginia requires 12 inches of cover so additional topsoil or mulch must be placed when drip tubing is installed at shallower depths.

Low Profile Mounds

The Wisconsin Mound, originally designed for use with septic tank effluent, has been modified when highly pretreated effluent is used (Wisconsin Mound Soil Absorption System, Siting, Design, and Construction Manual, Converse and Tyler, 1990, with updates in 2001). (Virginia incorporates this manual by reference to the 1990 manual in their regulations.) When drip dispersal tubing coupled with pretreated effluent (BOD and TSS <25 mg) is used, depths to restrictive layers such as a water table can be as little as 10 inches from the surface or 12" for non-creviced bedrock. In this situation at least 12 inches of sand capped with at least 12 inches of soil is required.

Typical components of a drip dispersal system are...

- Effluent pre-treated with an aerobic pre-treatment unit (activated sludge), a fixed film media (aerobic) system, or a system combining both of these approaches.
- A time dosed high head pump
- A filter after the pump (typically either a spin filter or disc filter, either of which needs to be routinely and automatically cleaned)
- A mechanical or solenoid valve mechanism to split the total flow into subflows going to zones – this allows use of a smaller pump. We shall refer to this as the zoner valve. (If

the number of orifices is small enough, and only one zone is used, there will not be a zoner valve.)

- A manifold line between the zoner valve and each drip zone. The manifold line splits the flows into laterals which for friction loss reasons are typically less than 600 ft. in total length. Laterals are comprised of runs which are the lengths of pipe joined at each end between to provide a continuous flow path between the input point and the exit point.
- A pressure limiting valve before each zone to prevent over pressurization of drip tubing and premature failure of orifices.
- A vacuum release valve to allow water to drain from the tubing without sucking mud into the drip emitters
- Drip tubing with emitters typically located every 2 ft. and typically with flow rates of about 0.5 to 1 gph.
- Check valves at the end of each lateral to keep flow from entering from other zones via the return line
- A return line to carry effluent back to the pre-treatment tank or pump tank so that flushing of the drip tubing occurs with a minimal velocity of 0.5 to 2 ft sec^{-1} depending on manufacturer's recommendations or state or local code.

HYDRAULIC DESIGN OF PRESSURE DISTRIBUTION SYSTEM

The concept of a low pressure piping system for dispersal of septic tank effluent is based on uniform distribution of the effluent across the trench area. This limits the operating pressure range to 2 to 5 ft. water head or about 1 to 2 psi. Lateral lengths are limited to about 50 ft. to control friction loss. Individual lines typically have throttling valves to off set head loss or gain associated with slope in the landscape.

Drip dispersal systems are far more robust due to the availability of pressure compensating tubing operated between about 10 and 40 psi. Non pressure compensating tubing can be used but there should be less than about 8 ft. of elevation difference in a zone to maintain a distribution variance of less than 10%. Especially with sites with more than 10% slope, we find systems with pressure compensating tubing are easier to design and install.

Consideration of Head Loss Due to Static Head and Friction Head Losses

The pump must be sized to provide an adequate flow at given pressure losses between the first and last orifice while maintaining pressure within the design range.

Static head loss (or gain) is the difference in elevation between the pump and the orifice in question. Obviously the system must be designed to deliver flow to all of the orifices in each zone and to deliver return flow back to the system;

Friction head loss occurs due to friction between the water stream and the surfaces it rubs against. The higher the flow rate, the higher the friction loss rate. The smaller the tube diameter, the higher the friction loss. Furthermore, substantial friction loss occurs across spin or disk filters, zoner valves, check valves, and other fittings. Fortunately several manufacturers offer spread sheets for calculating these losses. A fundamental understanding of hydraulics as well as

how drip dispersal systems are put together is imperative to put together a properly designed and installed system.

Finally after these losses are considered, the operating pressure of the system must be added to the friction losses to determine the operating pressure of the pump. Likewise flows must include any flows lost at the spin filter, the maximum flow in the largest zone, and the return flow to the pump tank.

Tubing Installation

Tubing should be rated for wastewater use. Tubing is preferably installed with a vibratory plow at a depth of 6 to 12 inches. It can be installed with a trencher or by hand but this is much more labor intensive in our experience. Tubing can be installed on the surface and covered with soil. Low profile mounds are a special condition of this where we placed the sand, place the drip tubing, and then cover it with soil.

Operation and Maintenance

The future of successful operation of these systems will include some type of mandated operation and maintenance program. It will also include installation by properly trained installers committed to quality workmanship.

First and foremost septic tanks and drain fields which require essentially no maintenance have ruined homeowner expectations. Until and after regulations are adopted requiring routine maintenance and inspection, homeowners must be educated that they have a custom built unique system that requires maintenance and inspection which will result in a routine operating cost. This education must be done by the original designer, by the regulator, by the realtor, by notice on property deeds, and by maintenance providers as the system moves from ownership by the original owner to subsequent owners.

Those who live with central sewer systems have a monthly operation and maintenance fee to pay for the capital and operating expenses for the sewage and collection system. (Unfortunately, this has been highly subsidized at the federal level in the latter half of the 1900's) resulting in unrealistic expectations even by current customers as these system age and must be maintained.

We who utilize onsite systems to live in more rural areas should expect to pay some cost to maintain our onsite treatment and dispersal system.

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ELIMINATION OF POINT DISCHARGE SYSTEMS BY CONVERSION TO ON-SITE TREATMENT SYSTEMS

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ABSTRACT

Point discharge systems PDS which utilized small treatment systems to treat flows of less than 1000 gpd. They may discharge to dry ditches, streams or even lakes. In Virginia, these systems are jointly regulated by the Department of Health and the Department of Environmental Quality.

There is a strong basis to require that these PDS sites be re-evaluated as part of the permit renewal process to determine if they can be converted to on-site treatment and disposal systems (OSTDS). PDS systems at individual homes are quite unlikely to be maintained on a continual basis resulting in the high probability that a significant number are discharging poorly treated non-disinfected effluent. Conversion to an OSTDS that includes aerobic pretreatment of effluent followed by either conventional trenches or shallow or surface placed alternative dispersal systems is far more robust than discharge directly into a stream with only disinfection via chlorination or ultraviolet light. Two examples of recent conversions are discussed.

INTRODUCTION

Ironically the small flow point discharge systems (PDS) are permitted under the Virginia Point Discharge Elimination System program co-administered by the Virginia Department of Environmental Quality (DEQ) and the Virginia Department of Health (VDH). Similar programs exist outside of Virginia. PDS are the systems of last resort when sites did not meet proscriptive VDH regulations for OSTDS at the time of permitting and are located beyond the reach of a centralized sewage collection and treatment system.

Most of these systems have been permitted prior to the late 1990's, a period since which many more options for OSTDS have been added to Virginia's Sewage Handling Regulations, primarily via General Management Procedures (GMP's). Virginia's Department of Health, Office of Environmental Health Services, many local VDH environmental health specialists (EHS's) and Virginia's very active private sector of earth science, environmental engineering professionals,

OSTDS contractors, and a wide array of manufacturers have been leaders in implementing many innovative alternatives for sites that did not traditionally meet prescriptive regulations.

As a result, sites which traditionally could not be permitted for OSTDS and may have a PDS, may indeed have a site that will meet requirements for an OSTDS with advanced secondary treatment coupled with an alternative effluent dispersal system.

EVOLUTION OF THE TREATMENT AND DISPERSAL CONCEPT

Historically on-site treatment disposal systems relied on pretreatment with a septic tank which results in about a 45% reduction in effluent concentration (Jantrania and Gross 2006). Flow through unsaturated soil was expected to provide the remainder of the treatment as well as the dispersal of the effluent. The proscribed amount of unsaturated soil that is required below the infiltration point varies from state to state, often dependent on what soils are locally available.

Recent advances in some of the more forward thinking areas of the country have led to utilization of pre-treatment with some form of oxygen addition whether passive (fixed film media (*i.e.*, sand filters, trickling media filters, peat filters, packed filters or textile filters) or active (aerobic treatment units utilizing activated sludge, sequencing batch reactors) or systems using a combination of these approaches.

With this additional pretreatment of effluent, research and application has indicated that the soil system moves from providing most of the treatment of the sewage stream to having more of a polishing function. As a result, the amount of oxygen transfer, the depth of unsaturated soil below the infiltration surface, and indeed the permeability needs are likely to be dramatically less. Hence soils with far more restrictive properties such as shallower depths to bedrock or water table, lower permeability or all of the above may be suitable.

Furthermore, this relatively “clean” effluent can be successfully applied with specialized drip dispersal tubing which is similar to drip irrigation tubing. This allows placement of the infiltrative surface much higher in the soil profile or indeed above it such as when a sand layer is added on top of the soil surface.

With these more sophisticated approaches, comes the challenge to develop systems that are robust and require less rather than more management. Never the less, management and monitoring of these systems is required. Development of management regulations and code requiring utilization of Responsible Management Entities (RME’s) has lagged but out of necessity is being developed – often at the local level before the state level.

SHORT FALLS OF PDS

PDS typically have the pre-treatment half of the more advanced OSTDS available today. They typically rely on chlorination or ultraviolet disinfection prior to discharge. Unfortunately many of the systems are not as advanced as current ATU or media filters on the market. Many if not most may not be adequately maintained, if at all. Indeed due to lack of regulatory effort (usually blamed on lack of budget), and the turnover of home ownership state and local records of these

systems may be poorly maintained if at all. Almost no testing for compliance occurs by our agencies given the responsibility to protect the environment. Operated in this environment, we can only expect that many systems are out of compliance.

Thus a commonality exists between advanced OSTDS currently allowed in the VDH regulations and the PDS systems – both need implementation of regulations requiring at least routine inspection by a Responsible Maintenance Entity. This must include a verification component.

TWO CASE STUDIES IN WEST CENTRAL VIRGINIA

Simon & Associates, Inc. was retained to provide annual routine testing services for two systems located in the Roanoke River Valley in Montgomery County, Virginia after the previous provider went out of business. One site had been previously operated as an experimental spray irrigation site by Virginia Tech under a research grant funded by Virginia Tech. Neither site met prescriptive regulations in the mid-1980's when the houses were built.

Both of these sites utilized an ATU that provided aeration via circulation of the water within the tank. This technology, while perhaps state of the art in the 1980's, is not today. Both systems were failing by a substantial margin to meet the requirements of the VPDES General permit for both total suspended solids and BOD.

THE MEDIA FILTER WITH CONVENTIONAL TRENCH SYSTEM SITE

This 5 acre lot has an intermittent mountain stream, a tributary of the Roanoke River, that runs through the lot. The area immediately around the house had limestone bedrock outcrops with shallow soils in between the outcrops. We were originally retained to replace the failing ATU.

In order to replace the ATU, we had to prepare a new PDS application as outlined in the Alternative Discharging Regulations for Individual Single Family Dwelling (12 VAC 5-640). This included identifying where the proposed discharge would be. As our surveyors marked the property lines so that the discharge could be properly shown, it became apparent that the active discharge point was at the lower property line on the stream – not in compliance with the permit. Furthermore, the conveyance line to that point, flexible corrugated pipe had come apart above that point.

After the property lines were marked, the hardwood vegetation suggested that there was a small area of deeper soils between the house and the stream. Backhoe pit evaluations indicated that soils at the installation depth were silty clay loams and clay loams which were underlain by clays and limestone bedrock. There was adequate area for conventional trenches for a five bedroom home (design flow 750 gpd) if a media filter was used. Using VDH GMP 114, an Advantex™ AX-20 system was installed following application for a Sewage Handling and Disposal permit (12 VAC 5-610-10 *et seq*). The permit was proposed and prepared by John J. Simon, a VDH authorized on-site soil evaluator (12 VAC 5-615).

The dispersal system is comprised of eight trenches three ft. wide and fifty ft. long with a total trench bottom area of twelve hundred ft². Trenches were placed on ten ft. centers. The total

drain field foot print was about 4000 ft². The trench bottom was installed at about 24 in. There was at least 12 in. of soil below the trench bottom as required by VDH regulations. A fifty percent reserve area was present above the field. Additional area located across the stream met requirements for an alternative drip dispersal system. This option was not chosen because the drip dispersal systems require more maintenance.

This system has been operating almost a year. It is monitored by Simon & Associates, Inc. with telemetry provided by the manufacturer. High flows on weekends during occasional social events have sent high flow notifications via email.

It is of interest to note that the area used was adequate for a three bedroom conventional OSTDS with a septic tank and low pressure piping dispersing to gravel trenches at the time the house was built (Due to regulation change, an ATU would be required today). Due to the prevalence of bedrock outcrops on much of the property, the difficulty of access with backhoes, and use of hand augers in gravelly soils, this site appeared unsuitable and was rejected in the 1980's. This is further cause for future review of sites as permits are renewed.

The house nearby lot which probably has a value in excess of half million dollars reportedly has a similar PDS system. This raises the question for all PDS permits: If a site for an alternative OSTDS is available, should a PDS permit automatically be re-issued without a new evaluation to see if there is a site compliant with current VDH Sewage and Handling Regulations?

SITE WITH AN AEROBIC TREATMENT UNIT FOLLOWED BY DRIP DISPERSAL

This site which is located on a high terrace of the Roanoke River along near an intermittent stream contained a PDS made by the same manufacturer as the PDS at the previous site. Again this system was well out of compliance with BOD and TSS values 50 to 100 per cent above the permitted values. When the experimental system was removed, a permit for conventional trenches was issued by the local VDH office however this system was never installed.

Simon & Associates, Inc. evaluated the same site and determined that a seasonal water table was located about twenty-four inches below the surface. Soils at the installation depth of twelve in. and in the horizon below were silty clay loam. Tight shale was located within forty eight in. of the surface and is the source of the shallow water tables throughout this part of the Roanoke River Basin.

The design for this site utilizes a Delta Whitewater D60 unit (designed for six hundred gpd flow) followed a pump tank and time dosed drip dispersal tubing placed at twelve in. This system has been installed by Simon & Associates, Inc. using a vibratory plow for placement of the drip tubing. Simon & Associates, Inc. is providing semi-annual service inspections of the system under contract with the owner.

FUTURE ISSUES

DEQ staff in the West Central Regional Office and VDH staff in the New River District recently developed a spread sheet list of PDS systems in the New River District. Of the forty systems in

the list, ten may not be currently permitted and about fifteen are in an application phase. A substantially greater emphasis needs to be placed on verifying compliance and permitting of existing systems throughout the Commonwealth.

Considering that the two replacement systems discussed represent twelve percent of the currently or formerly permitted systems in the New River District, there is a substantial possibility that as many as half or more may comply with current VDH Sewage and Handling Regulations if some combination of aerobic pretreatment and alternative discharging systems are used. This raises the following questions:

If the PDS permit has expired, should additional evaluation of soils be required?

If the PDS system is not functioning should replacement of the ATU and/or use of a permitted alternative dispersal system be required?

Is the fact that an ATU followed by an alternative dispersal system can be expected to cost twice as much as an ATU with disinfection and point discharge a reason to renew a Point Discharge Permit rather than require upgrade and conversion to a non-discharging system?

If new VDH Sewage and Handling Regulations require maintenance of alternative systems by an RME, will the PDS regulation similarly be amended?

SUMMARY

In their book *Advanced Onsite Wastewater Systems Technologies*, Jantrania and Gross ask the question “Is it possible for professional designers to confidently say that if some minimum amount of land area is available in relationship to expected flow rates on land that is suitable for home or business, an onsite effluent dispersal system can be design for any give soil and site condition?” They answer the question with a “Yes”.

We ask the question, if it is acceptable to send partially treated effluent to a point discharge, is it not better to apply the same water to existing or improved soils over an adequate area for it to be assimilated on site or to at least be converted to ground water perhaps seasonally seeping down slope provided this water meets minimal standards. After all, nature has groundwater seeps in foot slope positions in many systems.

The challenge then is to look at our programs and sites in the true spirit of the “Point Discharge Elimination” and to creatively provide alternatives while protecting public health and serving existing and future housing in areas with limited soil resources.

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THE STATE OF THE ART ON-SITE WASTEWATER DISPOSAL IN VIRGINIA

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KEY WORDS: septic system, aerobic pretreatment units, drip dispersal

ABSTRACT

Forty years ago, onsite wastewater treatment and disposal systems in Virginia and throughout the country were primarily septic tanks draining to a gravel bed or gravel trenches. In the last forty years advances include pre-treatment of sewage with aerobic treatment units with discharge to dispersal systems for polishing by the soil system as water is returned to the hydrologic cycle. The new approach is to move the primary aerobic treatment zone which was formerly the soil system back to a controlled aerobic environment. This allows more efficient use of increasingly limited soil resources. As these changes occur, new maintenance regulations and installer certifications will be required and are being developed around the country. This paper gives an overview of the types of onsite systems currently used today in Virginia.

INTRODUCTION

Twenty five years ago, I joined Virginia Tech as a Research Associate to conduct research regarding the effects of new septic tank effluent loading rates allowed for clayey soils typical of Ridge-Valley Physiographic Province of Virginia. Virginia was in the process of implementing the 1982 version of the Sewage Handling and Disposal Regulations promulgated by the Virginia Department of Health. These regulations implemented the concept that loading rates could be increased if effluent was equally distributed through out the drainfield.

Soon after arriving, we purchased a TR-100 lap top computer with 8 kb of random access memory. I contend that the concepts and implementation of on-site treatment and disposal have arguably changed almost as dramatically as the computers on our laps and desks. Yet some of our systems are still the same as we were using at that time.

THE PROCESS OF WASTE WATER TREATMENT

Wastewater treatment and disposal is first and foremost a process of recycling water and waste organic matter back into the environment. This process in some form occurs both on a primitive camping trip and in the most densely populated urban environments. On-site disposal systems lie in a range of the continuum between these two extremes. In the former, the density of waste generation relative to the availability of the earth's assimilative capacity is very low. In the latter it is very high. In the former, no technology is required, in the latter such as a metropolitan sewage treatment plant, very high levels of technology, energy, and management input is required to re-assimilate the waste into the environment with the least disruption of the

environment. In the former, land application is the basis of the recycling process. In the latter, the water and the waste it bears are often so highly treated that the water is returned to a stream as clean or cleaner than when the untreated potable water was first withdrawn from the same stream.

On-site wastewater disposal systems historically have been on been closer to the camping end of the continuum. However, as our use of water for domestic purposes has increased, as our population density as increased in unsewered areas, and especially as the quality of soils available to assimilate the water and waste back into the environment has decrease, our systems have begun to migrate towards systems that are more complex and incorporate many of the properties of municipal systems.

Wastewater treatment as typically practiced in an on-site environment consists then of two primary components:

- respiration of organic wastes back to primary inorganic components such as carbon dioxide, organic and inorganic nitrogen, and other less abundant but equally important trace elements and chemicals
- recycling of the water component of the wastewater stream back into the surface, subsurface and atmospheric water bodies.

These two processes are highly related in that if the degradation environment is aerobic, it tends to be much more efficient. If oxygen is lacking, it tends to slow dramatically sometimes resulting in organic matter accumulations and sealing of soil porosity. Hence if primary (septic tank) effluent is applied to poorly drained soils at a depth below the water table, the area needed for treatment may be much higher than if the effluent is applied to soils were oxygen is readily available. If organic matter is accumulating, and soil permeability is decreasing, then the potential for long term sustainability of a sewage treatment system may decrease.

Two interesting parallels have occurred in the last 50 years –

- The centralized sewage treatment community once relied on primary and secondary treatment followed by dumping water into streams to allow nature to finish the job. This resulted in eutrofication of large water bodies such as the Chesapeake Bay. In the latter half of the 20th century, the sewage treatment plants evolved to incorporate multiple sub-environments to enhance better oxidation of wastes, better removal of organic and inorganic nitrogen and phosphorus which can have tremendous impact on water bodies, and better processes for the settling or removal of the living activated sludge bodies from the water stream. One of the more interesting developments in wastewater treatment was the understanding that introduction of media in the treatment process such as trickling filters, sand filters, polymer sheets, *etc.* made the process more robust because microbial populations could colonize on these “bio-film” surfaces provided adequate oxygen was introduced into the water by aeration and supplemented by recycling of nitrate (NO_3^-) into anoxic zones.
- The on-site sewage treatment community which first relied on anaerobic septic tanks as primary treatment followed by some form of gravel infiltration bed placed in soil came to realize that if the untreated septic tank effluent could be evenly applied to the soil surface,

the soil treatment component of the system could be maintained indefinitely. An example is the introduction of low pressure distribution systems in Virginia regulations. This evolved in the last 15 years to the understanding the aerobic pretreatment of the effluent could dramatically enhance the ongoing ability of soils to maintain a water transmitting capability comparable to its native water transmitting capacity. The final and most interesting evolution has been to use media in the treatment tanks to obtain advanced secondary effluent because again the presence of a “bio-film” surface results in a treatment environment that is more robust.

A deeper look at the evolution of the treatment processes leads us back to the “egg” before the “chicken” of our ever intensified loading of our recycling systems: the soil system is the original and most robust “bio-film” surface. Hence when applicable, the soil system has a very great potential to be a viable component of decentralized sewage treatment systems whether these systems serve one house, clusters of several, or even whole subdivisions and neighborhoods. This evolution is apparent as the EPA has developed a focus on decentralized sewer system as often being the optimal method of recycling wastewater rather than the collection of water from many water sheds and transmission via pipe line to one or more huge water treatment factories. This is in part because, while we have continually upgraded the “factories”, our transmission structure update has often fallen behind both the demand caused by growth and the decay caused by the aging of the construction materials present to a sometimes caustic environment.

THE STATE OF THE ART – PRETREATMENT

Pre-treatment of on-site disposal systems has dramatically evolved. It includes septic tanks, septic tanks followed by activated sludge aerobic treatment tanks, septic tanks followed by sand filters and peat filters, small sequencing batch reactors, multi-compartment tanks which have anaerobic and aerobic zones coupled with bio-film surfaces. Indeed a look at many of the commercially available systems illustrates that each of the systems can be viewed as the creator’s use of different approaches to achieve a similar goal in different capital, management, and regulatory environments. With this in mind, we will look at some of the systems present in the market place in Virginia today. Inclusion or exclusion of a particular product is neither an endorsement nor rejection of any given product.

ANAEROBIC PRETREATMENT COMPONENTS

The following components which may stand along or with other components typically operate in an anaerobic environment, in that oxygen is not typically introduced. An exception is that in some designs, air may be introduced in equalization tanks to keep the sewage “fresh”.

SEPTIC TANKS

Septic tanks have the primary function of causing the waste to settle into a predominantly liquid form that includes both a dissolved and suspended component. Typically much of the organic nitrogen initially present as waste proteins and urea is transformed into ammonium ions. Grease and scum floats to the surface. The effluent flows out of the tank by way of a tee that draws the waste water from a depth that is about 30% of the depth of the tank. This allows denser solids to

settle to the bottom as sludge. This sludge should be removed every 3 to 5 years or more often depending on the source of the sewage.

The typical retention time of a septic tank is designed to be about 48 hours of “design” flow. Because design flows are often 50 to 100% higher than real flows and because there are peak flows in almost all settings, retention is affected by the flow path and treatment can be improved by introducing multiple compartments to break up short-circuiting flow. This is one of the earliest “improvements” available to septic systems but is still not typically used.

TRASH TANKS

This relatively new term reflects the one of the primary purposes of septic tanks. However it is used to refer to what is essentially a septic tank placed before an aerobic treatment tank. Its purpose is to lower the maintenance requirements for aerobic treatment systems by improving the quality of the influent due to loss of the non-digestible components such as grit, plastics, fibers, *etc.* that may be flushed into the sewage system.

FLOW EQUILIZATION TANKS

Domestic sewage treatment systems are predominantly biological systems, even if they are only comprised of a septic tank and gravel filled trenches. Flow equalization tanks are over sized tanks which contain a timer activated pump. Their purpose is even out the peaks and ebbs of sewage flow to provide a more steady state banquet for the microorganisms. They can be used to spread the peak flow of a church with services one or two days per week so that it goes to a septic system in much smaller flows each day of the week. This allows a much smaller soil treatment area for treatment and transmittal of the waste water.

Likewise equalization tanks are used to even out the flows into aerobic treatment systems with similar flow patterns. We have recently used them in the design of an aerobic pretreatment system at a 30 lot trailer park where flow is expected to have an early morning peak for about two hours and an evening peak for about 2 hours. Secondary peaks on Saturdays and Sundays are also expected. Equalization will allow a more steady grown environment in then pre-treatment system. A secondary effect will be the spreading of the hydraulic load to the infiltration system. Again the bacterial communities living on the soil bio-film surface will have a more steady diet and the water can flow away at lower velocities allowing for better pathogen attenuation.

AEROBIC TREATMENT COMPONENTS

Aerobic treatment systems may be comprised of activated sludge systems, activated sludge systems with bio-film media surfaces, multiple pass bio-film media surfaces, single pass bio-film media surfaces, or some combination of the above: “Naturally occurring microorganisms are the workhorses of wastewater treatment. Sometimes mistakenly considered to be “merely bacteria,” the ecosystem of a suspended growth aerated treatment system includes bacteria, fungi, protozoa, rotifers, and other microbes.” Jantrania and Gross, 2005)

ACTIVATE SLUDGE SYSTEMS

Activated sludge systems typically have a trash tank described above. It is followed by a tank into which the influent wastewater is introduced. This section of the system has air introduced via aeration tubes, stones, bubbler heads, *etc.* Mixing is often achieved by manufacturer's placement and design of the air inlets. In other design approaches, air is introduced by the use of a circulating pump system which may draw air in with venturis or even passively. The key concept regardless of how it is achieved is that an environment is created which promotes the "activation" of a bio-mass or "sludge" comprised of micro-organisms that eat the organics in the influent waste and in turn eat some of the sludge.

A second zone, called a clarification zone is present in which the water suspension is allowed to quiet. In this zone, sludge settles out and any scum or suds floats to the top. Key issues are designing the system so that good mixing occurs, adequate oxygen is provided to allow most of the sludge generated to be consumed, flow is uniform enough that a relatively stable growth environment is present, and ironically that the system is not over-sized so that the bio-mass starves. Clarification is important because otherwise more of the activated sludge leaves the system than is desirable.

ACTIVATED SLUDGE ENHANCED WITH BIO-FILM SURFACES

A second group of systems has added the enhancement of a bio-film surface that is suspended in the activated sludge environment. These systems may include the "trash" tank component within a compartment of a larger tank or the trash tank may be in the front. The bio-film surface has the purpose of providing a larger surface area for colonies of bacterial to live on – thereby possibly making a more robust environment to handle the natural peaks and ebbs in flow and nutrient make up of the wastewater.

SINGLE PASS MEDIA FILTERS

The first single pass media filters were arguably the gravel filled trenches or pits associated with septic systems. From this grew the concept of placing a sand layer in tanks underlain by a gravel layer. As sewage effluent flowed through the sand reduction in wastewater strength occurred. Obviously from previous discussion it is possible to ascertain that the better the spreading of the effluent across the surface and the lower the loading rate, the better the treatment might be expected.

Evolution has occurred down two trains –

- utilization of septic tank effluent dosed peat filters which are single pass or utilization of sand filters with single pass of dosed septic tank effluent
- utilization of multiple pass filters.

The limitations of single pass filters can include in adequate introduction of oxygen and over loading especially if distribution is not enhanced with pumping or some other dosing mechanism. An interesting mechanism that is utilized for this enhancement is the "ecoflow" peat filter which uses a gravity flow loaded "tipping bucket" to alternately dose two sides of the filter.

MULTIPLE PASS FILTERS

The earliest multiple pass filters were probably re-circulating sand filters. Sand filters included a pump which returned a portion of the effluent to the filter surface resulting in many passes and many opportunities for oxygenation of the environment. Higher loading rates per area of sand surface can be used and a more uniform effluent quality is expected.

Innovations in the last couple decades have included substitution of other types of media such as fly-ash, hanging textile sheets (*i.e.*, Orenco's Advantex system), tire chips, and wood chips. Some of the peat systems are also operated as multiple pass systems by re-circulating part of the effluent back through the system.

Because of the filtrative nature of these systems, clarification typically occurs as the water passes through the media.

SEQUENCING BATCH REACTORS

Sequencing batch reactors work by having different sequential phases within a treatment unit. The phases include the loading phase which may be an aerobic or anaerobic phase, an extended aeration phase, and a clarification phase when the aeration is turned off.

Characteristics of SBR's include the potential for treating some types of wastes in which part of the waste stream responds better to anaerobic treatment and part responds better to aerobic treatment. They can be effective for denitrification due to the potential for the system to go anaerobic during both the loading and clarification phases. An example of this approach is the Aquarobic system.

PRETREATMENT SUMMARY

As there are several ways to cook dinner, there are several ways to treat waste water. Some have been developed to require the least amount of attention and maintenance. Others require more maintenance to maintain their optimum performance. By manipulating the presence of anaerobic and aerobic zones, much of the nitrogen present as nitrate can be removed, provided there was adequate alkalinity and oxygen present for the ammonium to be nitrified. Good pre-treatment systems result in substantial reductions of pathogens. All of the systems discussed have one thing in common, they reduce the organic matter present in the water but they do not significantly reduce the volume of water that must be recycled into the environment. Likewise as environmental or maintenance conditions develop, any of the systems can have a substantial variation in effluent quality. These issues will be addressed in our final section on maintenance.

WATER RECYCLING

Twenty-five years ago, we who were enlightened began to refer to septic systems as on-site wastewater disposal systems. The problem with this term is the word disposal for it illustrates the central problem with early central sewage and on site sewage disposal approaches – both ultimately are part the water cycle.

Decentralized systems (both individual and group) have the potential to return water to the local water shed from which it came, albeit we may not return it directly to the same aquifer from which a well extracted it.

More importantly, use of the soil system has the benefit that the soil (especially coupled with vegetation – either the root zone or plant surface in the case of spray irrigation) has the potential to highly filter and attenuate the final impurities in the treatment system effluent. The pretreatment systems discussed above have the primary benefit of being a very efficient manner of dramatically reducing the organic load so that the infiltration system becomes more of a polishing and transition system. This allows use of smaller volumes of soil because the quantity of oxygen which must be transmitted through the soil by passive aeration is dramatically reduced.

Most modern on-site treatment regulations place a large portion of their focus on soil properties ranging from color, to texture, to estimated or measured permeability to thickness of the solum or regolith. Water moves primarily in response to the force of gravity and secondarily in response to capillary action (*i.e.*, the wicking of water up the surface during drying).

Successful recycling (or disposal) of the water is dependent on having an adequate capacity for assimilation of the water. If the water flows through an unsaturated environment, the soil environment is likely to be aerobic resulting in maintenance of soil porosity and maintenance of an environment in which water flows through smaller pores resulting in better physical filtration of pathogens such as bacteria and viruses.

INFILTRATION SYSTEMS

Infiltration systems fall into four categories:

- Subsurface infiltration trenches
- Subsurface infiltration via drip dispersal tubing
- Fill systems placed on the soil surface
- Spray irrigation systems.

The first three systems have the shared attribute of introduction of the effluent below the vegetative surface. While they are allowed in some states as a mechanism for single family homes, they are rarely used in Virginia because they are considered a point discharge system in this state. This concern about aerosol or vectors available to humans by vegetation contact is the driving force. This said, this approach has been successfully utilized in Virginia but will not be further discussed due to its limited used.

SUBSURFACE INFILTRATION TRENCHES

Gravel filled trenches or beds are the oldest and most widely used infiltration mechanisms. Poor distribution of effluent associated with gravity distribution can result in localized overloading and early failure of trenches especially when organic rich septic tank effluent is used. Dosing of gravity type trenches causes “enhanced flow” or distribution and is likely to improve

performance provided groundwater intrusion into pump chambers does not occur. Low pressure distribution of STE was introduced in Virginia code in 1982 along with the allowance of an area reduction especially in soils with more clay which otherwise have design loading rates which decrease exponentially. This reflects the understanding that full utilization of the soil bio-film surface results in less sealing of the inherent permeability.

Recently, gravel substitutes have been allowed. To make these cost effective, the controversial practice of an area reduction without use of enhanced flow or LPD has been allowed by the VDH. This author believes that LPD or aerobic pretreatment should be used if area reduction is to be allowed.

DRIP DISPERSAL SYSTEMS

Utilization of drip dispersal tubing placed at shallow depths is an evolution of the LPD concept. Current practice is to require pre-treatment and filtration of the effluent to prevent clogging of drip emitters. Systems are operated at a pressure of ranging from 10 to 60 psi. The drip tubing is closely spaced to more closely simulate natural rainfall infiltration. In this approach, we have moved to the concept of a foot print treatment area. Maintenance of both the pre-treatment systems and the dispersal systems is critical.

MOUND OR FILL SYSTEMS

The concept of placing what is essentially a single pass sand filter on top of an existing soil surface where only a limited thickness of suitable soil was present was first introduced by researchers at the University of Wisconsin. Virginia incorporated this design approach utilizing septic tank effluent in the 1982 regulations. Subsequent experience has led to the evolution in Virginia and Wisconsin of using drip dispersal of pre-treated effluent in low profile mounds. This allows utilization sites with steeper slopes. The most recent emphasis has been on spreading the load across the slope rather than stacking it down the slope. The term linear loading rate has now been incorporated into Wisconsin Code and practice and is part of careful conceptual evaluation if not code in Virginia.

These systems can allow us to approach nature in the most limited conditions if we achieve pre-treatment, disinfection (*i.e.*, uv light or chlorination) and dispersal into the sand filter. If the above goals are achieved, but water seeps at the toe of the mound, the water may arguably be ground water seeping at a rock or less permeable soil layer contact just as wet weather springs are part of the natural environment.

INSTALLATION AND MAINTENANCE OF ADVANCED TECHNOLOGY SYSTEMS

Virginia has been on the forefront of introducing the use of pre-treatment systems (but by no means the first.) As well they are on the forefront of using alternative infiltration via drip dispersal systems. Two key issues are evolving – quality of installation and need for routine maintenance. While the Virginia Department of Health expresses concern about both issues, regulations are still lagging behind innovation. This problem is widespread in the industry. Industry and government are cooperatively attempting to address these issues with new

regulations requiring maintenance. While state-wide regulations are still being written, some local governments have enacted regulations requiring periodic maintenance of advanced wastewater treatment and dispersal systems.

Parallel to this course is an industry led push to require adequate training and certification of installers of on-site systems. It is safe to say that in much of Virginia, 50 to 80% of new systems have either an aerobic treatment unit and/or drip dispersal component. Many systems include micro-processor controllers, complex tubing layouts requiring quality workmanship and specialized training and knowledge. The Virginia On-site Waste Water Recycling Organization is cooperating with the Virginia Health Department and the Virginia Department of Professional and Occupational Regulation to establish minimum training and knowledge requirement for contractors.

Coupled with these developments, is the development of companies or consultants who are beginning to offer service and maintenance contracts to homeowners. Ultimately service will be sourced from a continuum ranging from the private sector to responsible maintenance entities (RME) which can be privately owned or quasi-government agencies.

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VOWRA – A PLAN FOR REGISTERING COMPETENT INSTALLERS OF ONSITE SYSTEMS

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ABSTRACT

The Virginia Onsite Wastewater Recycling Association was originally incorporated in 1996. It is comprised of members from the design, manufacturing, installation, and regulatory communities who are committed to furthering the quality and performance of onsite waste water treatment and disposal systems in Virginia.

Currently the only requirement for certification of installers of septic or more advanced onsite wastewater treatment and dispersal systems (OSTDS) are requirements imposed by manufacturer's of proprietary systems. Most manufacturers require that installers have a Virginia Contractor's License with either a SDS or HH certification. No training or continuing education is required, nor does the Virginia Department of Health require the contractor's license number on the contractor's completion statement which must be completed before an operations permit is issued. Therefore, many systems are installed by individuals or companies who do not have a contractor's license.

Increasingly complex OSTDS require specialized skills such as reading blue prints, installing and setting up programmable logic controllers (PLC's), properly installing often several hundred plumbing and electrical connections in relatively hostile soil and wastewater environments and then starting these systems and teaching the homeowner how to operate them. The \$3000 septic system is now more often than not replaced by the \$8000 to \$20,000 dollar aerobic, fixed film or packed bed filter treatment system followed by an enhanced effluent dispersal system.

Concern is being voiced across the country about the need to adopt maintenance and performance requirements for these systems that reflect the complexity of the system and its operating environment. Just as important is adoption of a minimum training standard and a readily available source for that training.

Effective January 2007, VOWRA plans to publish a list of installers who have voluntarily undertaken training regarding the design and installation concepts of OSTDS ranging from septic systems to more advanced systems. Manufacturers are indicating a willingness to require that installers of their proprietary systems demonstrate a minimum level of training such as that provided by VOWRA or community college courses being developed in conjunction with the Virginia Department of Health.

This is a stop gap measure until more formal regulation can be implemented hopefully by the Department of Professional and Occupational Regulation (DPOR) the agency that licenses contractors, soil scientists, geologists, and engineers in Virginia.

INTRODUCTION

Throughout the United States increasingly complex onsite wastewater treatment and dispersal systems (OWTDS) are being designed to address the need to use increasingly limited soils. When these systems were historically septic systems, they were relatively easy to install and were relatively forgiving even if workmanship was poor. With the increasing level of complexity, the need for competently trained and licensed individuals is evident, especially when some contractors' work begins to routinely fail often even before full time occupancy of a house.

EDUCATION EFFORTS

Government

The Virginia Department of Health established the Virginia Center for Onsite Wastewater Training in Blackstone, Virginia. This is a collaborative partnership between VDH and Southside Virginia Community College (SVCC). The Center's stated mission is to "Educate and train practitioners and others in the practices and technologies essential to assure long term sustainability of onsite wastewater systems, ground and surface water quality and public health."

The site will be providing both training conferences on a regular basis to VDH staff, Authorized Onsite Soil Evaluators, Professional Engineers, contractors and other interested individuals. A demonstration site is being completed where vendor donated and installed waste disposal systems can be installed.

VOWRA

Parallel to this effort is the action taken by the VOWRA to begin to provide a minimum level of training for installers in the form of a Pre-conference Education Day at its fall conference. More than forty installers paid to attend the A to Z Course taught by nationally recognized Jim Converse, Ph.D. This course has been provided at other times in 2006 and 2005 by VOWRA at sites throughout Virginia to provide a basic level of education for those involved in the design

and installation of onsite systems. More than 135 installers have attended the course in the past 12 months

Both VDH and VOWRA and those that have attended courses here and in North Carolina addressing these issues are to be commended for the steps in this direction. By beginning to provide a mechanism for voluntary education, the local leaders in the onsite industry who find themselves overwhelmed by or in need of additional education are beginning to set a trend and to recruit other competitors to join this education effort.

Creation of this ground swell transfer of knowledge has the potential to put a positive spin on licensing requirements which are needed to prod the less proactive members of the construction community to come up to speed.

THE VOWRA REGISTRATION PROGRAM

Many if not most of the manufacturers and distributors have some minimal level of training for installers using their product. This is necessary for protection of their brand and successful deployment of it. Beginning in January 2007, VOWRA will publish a list of contractor members who have attended it's "A to Z Course" and who have passed the subsequent test. The purpose of this program is to be as inclusive as possible and to help lift our industry up to a new standard from within.

Manufacturers of many of the OSWTDS products in Virginia are supportive use of the voluntary registration program and have indicated that they are willing to require this minimal level of training before selling their product to installers. In parallel many of the designers of OSWTDS be they authorized onsite soil evaluators or professional engineers are beginning to specify that installers will have taken the A to Z class or have equivalent training.

This program is being rolled out in January 2007 by VOWRA with the sincere goal that it is only a step towards changes in regulations by DPOR that require additional training education and continuing education for installers of OSWTDS.

WHAT IS AT STAKE?

Yesterday's septic system cost \$2000 to \$5000 dollars. Poor workmanship could result in problems but often it was a matter of re-grading a site or rewiring a pump. Poor workmanship on today's \$15,000 to \$25,000 alternative systems can quickly lead to wholesale failure of a system installed on a very limited site.

Certification is needed first and foremost to protect the homeowner and future owners of the system. It is also needed to protect the environment. When these systems are not properly installed, surfacing of partially treated sewage can be commonplace until repairs are made. If improperly installed, repairs can often cost as much as 50 to 100% of the original construction cost.

Finally, our industry is in a state of dramatic change in response to the advancement of science, technology, the creative application of new products and approaches and most importantly, EPA's recommended management guidelines for onsite decentralized wastewater systems. With such forces of change, it is imperative to keep our eye on the long view and to work through the regulatory and other changes that must be made to ensure the success of the onsite waste water industry in its mission to provide sewage disposal services to homes beyond centralized sewer systems.

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NITRIFICATION AND DENITRIFICATION IN SMALL ONSITE WASTEWATER DISPERSAL SYSTEMS

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ABSTRACT

Sewage is relatively rich in organic nitrogen present primarily in proteins. As the organic carbon in sewage is converted to carbon dioxide, water, and more microbes in the digestion process, much of the nitrogen is converted to ammonium (NH_4^+). Some is subsequently converted to nitrates if adequate oxygen is present. These processes occur at some point whether the sewage is pretreated with an aerobic treatment unit or if it is primarily treated after septic tank effluent is applied to the soil.

A substantial amount of nitrogen is lost as N_2 gas in the process known as denitrification. This is desirable if nitrogen is not being recycled through an agricultural production system. Nitrogen not lost as N_2 gas is typically converted to nitrate (NO_3^-) which is soluble in groundwater and can pose a public health threat in high concentration. With an understanding of the nitrogen cycle, some parts of the treatment process can be tailored to promote denitrification.

INTRODUCTION

In the nitrogen cycle, plant useable nitrates or ammonium are produced by nature and by man with industrial processes. Nitrogen is one of the fundamental building blocks of proteins. Hence wastewater from our homes contain a substantial amount of organic nitrogen. Typical organic nitrogen (TKN) in raw wastewater has been reported to be in the range of 60 mg L^{-1} . Septic tank effluent has a typical concentration of 40 mg L^{-1} of ammonium but typically does not contain nitrates due to presence of an anaerobic environment. Effluent from a media filter (an aerobic environment) can be expected to have 15 to 30 mg L^{-1} of total N present as nitrates and 0 to 4 mg L^{-1} present as ammonium (Jantrania and Gross 2006).

Because we have the goal of not degrading groundwater, Virginia encourages designs especially for flows in excess of 1000 gpd which will result in an average discharge across the dilution area of $<5 \text{ mg L}^{-1}$ after nitrogen removal by plants and by denitrification and after additional dilution by rain water (Virginia Department of Health GMP 101 1999). We will discuss mechanisms which affect the nitrate and ammonium loading that leaves the footprint of a soil dispersal system.

CONVERSION OF ORGANIC NITROGEN IN WASTEWATER TREATMENT

Cellular nitrogen is readily mineralized to ammonium + other cellular nitrogen absorbed by other microbes even in an anaerobic environment such as a septic tank or trash tank. Hence nitrogen in raw waste water or septic tank effluent is often reported as total Kjeldahl nitrogen (TKN) which includes the ammonium fraction.

Depending on the process this wastewater either moves to the soil system for aerobic treatment or to an aerobic pre-treatment unit (or media filter). In the soil environment, nitrifying bacteria convert the ammonium to nitrate if the environment is not too acid. If the trenches are placed below the root depth, much nitrate may leach. However in the presence of dosing of septic tank effluent, soil may become periodically anaerobic as new waste is applied. In this scenario, some denitrification is expected to occur due to absence of adequate oxygen as O_2 . In this situation, denitrifiers may use the oxygen present in the NO_3^- anion resulting in release of N_2 gas. Even though we may expect as much as 50% loss of nitrogen (or higher levels) in some natural situations, this is a difficult environment to control.

MANAGING THE NITRIFICATION AND DENITRIFICATION PROCESS IN MEDIA FILTERS AND AEROBIC TREATMENT UNITS

There is much greater opportunity to create an environment favorable to denitrification with a wastewater treatment system. For denitrification to occur, the following steps must occur:

- Conversion of organic nitrogen to ammonium
- Conversion of ammonium to nitrate – requires presence of some alkalinity to buffer the acidity created by this reaction
- Presence of an anaerobic environment with degradable carbon and facultative microbes seeking a source of oxygen. Note that these may be localized sites.

PRACTICAL WAYS TO ENHANCE THE DENITRIFICATION PROCESS

- Have a trash tank which is anaerobic due to excessive presence of partially treated carbon rich waste
- Recirculate a portion of effluent from a secondary treatment system such as a peat filter, aerobic treatment unit, or other media filter back to the trash tank
 - Caution, it is possible to overload the hydraulic capacity of some clarifiers if too much effluent is sent back through the trash tank (which then flows to the aerobic treatment unit)
- If the water source is acid ground water it may be necessary to add lime to the wastewater stream to get higher levels of conversion of ammonium to nitrate so that denitrification can then occur – this is a common problem in the Blue Ridge and Piedmont regions of Virginia.
- It may be possible to dose nitrate rich effluent through constructed wetlands to provide a mechanism for denitrification.
- Utilize less air in the secondary treatment process so that nitrates are denitrified within the unit. (Some manufacturer's designs have utilized this approach.)

NITROGEN REMOVAL BEYOND THE TREATMENT UNIT

Nature typically hungers for a source of nitrogen as plants grow. If effluent is dispersed in the upper part of the root zone, nitrate and ammonium present is much more available for plant uptake. Hence utilization of drip dispersal, especially where high levels of removal are critical may enhance removal. Harvesting of grass for use as forage can further remove nitrogen from the system.

Nature can include some additional natural surprises. The Blue Ridge Mountains often have wide fertile valleys which shallow perched water tables. On one farm we worked on many years ago, there was a high nitrate concentration below areas where unlined chicken houses had once stood. Groundwater flowed towards a natural bog associated with a creek at the valley bottom. In this environment, we were able to document the almost total removal of nitrates (using the change in the nitrate to chloride ratio) as water moved naturally through the ecosystem.

SUMMARY

Onsite wastewater treatment and dispersal systems vary dramatically in size and intensity of design and treatment. The fundamental principles of the nitrogen cycle relative to wastewater treatment should be understood and respected by designers and regulators. With this knowledge, when opportunity arises, simple modifications may allow efficient removal of nitrogen that otherwise might unnecessarily degrade water. A goal of total removal is unrealistic. A goal of appropriate removal for the environment and complexity that the system will be placed in is admirable.

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