JURIED ARTICLES

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A Rational Method for Determining Design Flows for Cluster Systems

GIS Mapping of Land Application Sites for Septage Management in Mahoning County, Ohio

Volunteer Teens Get Down and Dirty
North Carolina’s Less Fortunate Get Help
From the Editor...

Dear Small Flows Readers,

Welcome to the Fall/Winter 2007 issue of Small Flows Magazine. The issue has a number of interesting articles that I think you will find useful.

Our staff writer, Caigan McKenzie, presents a story about a unique volunteer program by the North Carolina Rural Community Assistance Project that brings volunteer teens from all over the country to help low income people who need septic systems repaired or installed. See this inspiring account on page four. On page eight, Mark Gross, P.E., and Terry Bounds, P.E., champion their view that water softener backwash brine stresses septic systems. They suggest simple and inexpensive discharge alternatives in the “Small Flows Forum.”

Also, this issue features three peer-reviewed articles: one about the performance commercially available chlorine and ultraviolet disinfection units, one about a method for determining the design flow of cluster systems, and one about GIS applications to map isolation distances for septic management.

Our products list on page 35 features materials related to these articles. The National Environmental Services Center offers products free or at low cost. And our technical assistance team remains ready to answer your questions about these stories or other information you may need. Please call 1-800-624-8301 where engineers Clement Solomon, Jen Hause, Zane Satterfield and our team of specialists can provide you with assistance, solutions, and knowledge if you ASK.

Erratum... 

Due to a printing error, an incomplete version of the juried article, “Effects of Embedment on Flow Through Sand Columns,” appears in the Small Flows Magazine Spring/Summer 2007 issue. The complete article is available in the online issue, which can be viewed and downloaded at http://www.nesc.wvu.edu/nesc/sfq_2007.htm.
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By Mark Gross, Ph.D., P.E. and Terry Bounds, P.E.

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Volunteer Teens Get Down and Dirty
North Carolina’s Less Fortunate Get Help

by Caigan McKenzie,
NESC staff writer

Some might find it odd for teenagers to trade a week of lazy summer days for labor-intensive days, complete with ticks, poison ivy, snakes, spiders, wasps, and hornets. Possibly more peculiar is their willingness to pay approximately $300 for the experience. But Rich Holder, community resource coordinator for North Carolina Rural Community Assistance Project (NCRCAP), doesn’t find it at all odd. This summer, like last summer, Holder will work with teenagers from across the country who have eagerly volunteered their time and money to hand dig and install septic systems for low-income families in western North Carolina.

Founded in 1987, NCRCAP is a non-profit organization that assists rural North Carolinians with water, wastewater, and housing issues through a variety of outreach programs.

Western North Carolina Septic Repair Program

One of NCRCAP’s programs is the Western North Carolina Septic Repair Program, which is committed to protecting public health, the environment, natural resources, and addressing poverty issues in western North Carolina. It was founded in 2006 after surveys conducted by local county health departments and the North Carolina Wastewater Discharge Elimination (WaDE) program showed a high incidence of wastewater treatment violations.

“I was part of the original survey team,” Holder said. “We went door-to-door to see if homeowners had a system. If they did, we dye tested it to make sure it was working properly. What we found was that many of the surveyed homeowners had failing septic systems or were straight piping their waste to local streams. In many cases, the homeowners were very low-income, unable to afford repairs, and lived on sites that were inaccessible to a backhoe.”

One of the surveyed homeowners lived in a dilapidated home with raw sewage flowing beneath it. Although the family was awarded a grant to purchase a mobile home to replace their current home, the grant was not enough to cover the cost of a septic system. The Land of Sky Regional Council of Governments, a local government planning and development organization that was instrumental in securing the grant, asked NCRCAP for help.

After surveying the site and recommending a pump system for the property, Holder asked the Emanuel Lutheran Appalachian Servant Event organization, which, through its volunteers had been repairing homes and septic
systems for more than 25 years, if they could hand dig and install the system to save the homeowner labor costs. Emanuel Lutheran agreed and spent one week of almost continuous digging.

**Septic Repair Program Structure**

“We (NCRCAP) see this program as Rich’s brain child,” Sharon LaPalme, associate director of NCRCAP, said. “Rich was involved with the county-wide surveys that occurred in the late ’90s. Since then, it has been his goal to see the long-term follow-up to those surveys. And, it’s where Rich’s skills and knowledge lie. He knows the residents, so he is very good at working with them; he has the needed technical expertise; and he has a lot of contacts in the wastewater industry.”

The septic repair program brings together funding groups, regulators, housing agencies, material manufacturers, and volunteers, all working together to assist low-income homeowners with their wastewater needs. To be eligible for the program, households must meet certain requirements, such as financial need and have a wastewater violation that is a threat to public health. Financial qualification is done through the Land of Sky Regional Council of Governments and community action agencies, while Holder knows which households have the greatest need for repairs.

“Many of the contacts that Rich made when he worked as an environmental health specialist donated materials to the septic repair program,” LaPalme said. “He has put together an incredible basket of materials free of charge.”

Available donated materials play a key factor in determining the type of system to install. Because Holder is a registered sanitary and an environmental health specialist, he is authorized to design and permit septic systems with the state of North Carolina Department of Environment and Natural Resources. In helping design the systems, he ensures each meets applicable onsite wastewater and plumbing regulations. But he is not a county employee, so he works with the county health department to permit the systems and to have the systems inspected.

To get local and national companies involved, Holder sent them a letter explaining the septic repair project. “Along with the letter, I included pictures of the kids digging,” Holder said. “The companies all jumped on board. A lot of them are friends of mine, but I think it was the kids volunteering their summer vacations and their own money that got everyone really interested in wanting to help.” Funding for materials and equipment that are not donated is provided through housing and watershed protection programs, such as the Community Foundation of Western North Carolina, Pigeon River Fund, the North Carolina Clean Water Management Trust Fund, and WaDE.

Volunteers for the program come from two youth, faith-based groups: the Emanuel Lutheran Church Appalachian Servant Event, and ReCreation Experiences, which includes a variety of religious denominations. “All of these kids are real heroes. They spend summer vacation
“Sometimes homeowners will get in the trenches with us,” Holder explained, “but mostly, they are too elderly to do that. It’s not a requirement that homeowners help.”

One homeowner who did help dig commented, “I can walk through my backyard now without stepping through that mess. As for the kids, it is great to see so many people who really care come together, especially young people.” Many homeowners will show their appreciation by supplying cold drinks and treats.

After installation, Holder teaches the homeowner, through literature and explanation, how to maintain the system.

**Obstacles**

An abundance of insects and poison ivy are not the only obstacles volunteers face. Each system has had overwhelming problems, according to Holder. He points out that he can’t always prepare for these problems, because he doesn’t know what he is going to find until they start digging. And digging has been extremely difficult because of the recent, severe drought in western North Carolina. “It’s like trying to dig through cement,” Holder said. Since conventional machinery cannot maneuver onto the sites, hand digging is the only option. Digging is also hindered by rocks and roots, and “we somehow find water and electric lines in our path that are not supposed to be anywhere near where we are digging,” Holder said.

Plumbing has also been an issue. In some cases, houses need to be replumbed because the only place that a septic system can be installed is at the end of the house opposite where the plumbing is located. This isn’t the only reason for replumbing. In some cases, plumbing fixtures have fallen into such disrepair that they could cause a hydraulic overflow, causing the newly installed system to fail.

Holder has also experienced electrical problems. “We had a pump system, but not enough power going to the circuit breaker box to power the pump,” Holder said. “We had to bring in an electrician to see if we could put in a larger circuit breaker box.”

“For some of the homes, it is as if there wasn’t a system there to begin with,” LaPalme said. “It involves installing all of the components: a septic tank, sometimes a pump tank, and, of course, a drainage field.” None of this deters Holder. According to LaPalme, Holder works more than 70 hours a week to get the work done.

**Silver Lining**

Occasionally, a home will not need much done to fix the wastewater violation. One home, for instance, had a septic tank that straight piped into a ditch that went into a creek. The upside of this story is that there...
also was a mobile home on the property that was no longer being used. It was conveniently located downhill from the house and already had a permitted septic system that would meet the needs of the homeowner and could be used as a permanent system.

“We just ran the pipe from the septic tank at the owner’s house to the permanent septic system [at the trailer] and hooked to it,” Holder said. “We found a way to use something that the owner already had rather than installing a whole new system to get him out of straight piping.”

**Septic Repair Program Grows**

In 2006, two households benefited from the program. This year, however, the goal is set for 10 households. Each site will have approximately 40 volunteers who will work between eight to 10 hours a day for five to seven days. Some of the repairs will include systems that require pumps, alternating valves, tanks, aggregate, pvc pipe, and filters.

“We don’t take into account how much a faulty septic system affects the community until we are standing in it only five feet away from a school playground,” said volunteer Bethanie Gallagher. “Many hours of hard work went into putting a dent into the problem.” The system that Gallagher is referring to is one slated for repair this summer that is located just 20 feet from the swing set of an elementary school.

“On the day that Rich went to do a site visit, he found the elementary school kids running around on the failing drainfield trying to catch the butterflies that the effluent and sewage attracted,” LaPalme said. “This system was clearly a threat to public health.”

**Teens Go Beyond Septic Repairs**

Volunteers not only repair septic systems, they also paint, repair bathrooms, roofs, and floors. “They are willing to do whatever it takes to make the homes a safer and nicer place to live,” Holder said. “They are truly amazing kids!” The teens have even gone beyond repairs in some cases to make the homeowner’s life more pleasant. One homeowner, for instance, had commented that she spent most of her time alone. When the teens found out it was her birthday, they threw her a celebration. “I really missed all the company I had that week, and the birthday celebration was very special to me,” the homeowner said.

**Request for Help**

“We still are in great need of donated materials to be able to complete these projects,” Holder said. “We would like to invite all organizations to join us by collaborating with agencies committed to improving water quality, public health, and helping those in greater need than themselves.”

For more information, contact Holder at rholder@ncrcap.org, LaPalme at slapalme@ncrcap.org, ReCreation Experiences at www.recreationexperiences.org, and Emanuel Lutheran Appalachian Servant Event at (828) 273-7669, and Terrell Jones at Terrell.Jones@nc -mail.net.

**Organizations that have supported the project through volunteer labor, funding, or donation of supplies are:**

- SE Rural Communities Assistance Project
- NC Rural Communities Assistance Project
- NC DENR Wastewater Discharge Elimination Program (WaDE)
- ReCreation Experiences
- Emanuel Lutheran Church Appalachian Servant Event
- Community Foundation of Western North Carolina
- Land of Sky Regional Council of Governments
- Watauga Avery Mitchell Yancey Community Action Agency
- Pigeon River Fund
- Western Carolina Environmental Inc.
- Local Health Departments (Buncombe, Henderson, Jackson, Madison, Yancey)
- Ring Industrial Group
- Infiltrator Systems Inc.
- Carolina Aerobic Systems
- Mariner Container Corp
- Mace’s Back Hoe Services
- AAA Water Flowrite Inc. (Plumber)
- Kingsway Ready Mix Concrete Inc.
- Zoller pumps
- The North Carolina Clean Water Management Trust Fund
- KDR Services (Electrician)
Water Softener Backwash Brine Stresses Household Septic Tanks and Treatment Systems

It is a fact that water softener brine regeneration discharges change the consistency and chemistry of the wastewater stream in ways that pose a problem for onsite treatment systems and the dispersal field. Studies have shown that water softener brine regeneration wastes not only harm the flora and fauna in the wastewater treatment system, they can also cause the septic tank itself to discharge greater concentrations of solids, grease, and oil into the dispersal field.

Since the purpose of the septic tank is to separate the solids and the fats, oils, and grease (FOG) from the liquid, discharging mostly dissolved organic matter and nutrients, the discharge of solids and FOG into the drainfield will cause the soils to plug resulting in an expensive drainfield repair. The concern is not the softened water nor whether or not sodium salts affect soil infiltrative capacity or long-term acceptance rates (LTARS). The concern is the high concentration of chloride salt in the backwash brine from softener regeneration. Regenerate brine is recognized and typically classified as a “salt-laden water, free of contaminant” and, thus, does not need to be discharged into biological wastewater streams.

Salt Stratification Inhibits Tank Performance

Research performed at the National Sanitation Foundation (NSF) used complete-mix aerobic treatment units, where the water softener backwash brine was introduced to a system that is completely mixed. Septic tanks were not part of the NSF study, and it would be misleading and scientifically inappropriate to directly compare any complete-mix aerobic process to a passive anaerobic process. Studies with septic tanks, which are designed to be quiescent by nature, have shown that the high concentration of salt introduced by slugs of backwash brine cause salt stratification in the tank, which inhibits the ability of solids and FOG to stratify.

The result is that the salt water dives to the bottom of the tank occupying space that is designed for the settling of heavier solids. In addition, the sludge in a septic tank is mostly liquid with a density very near that of the clear zone. The heavier salt water can actually lift the sludge from the bottom of the tank, displacing it and...
washing it into the downstream components such as an ATU, media filter, or the soil dispersal field. As a result of its density, the salt-laden brine competes with the sludge to occupy space at the bottom of the tank, effectively reducing sludge storage volume. This could increase sludge pump out frequency or allow the sludge to be carried out into the soil dispersal area.

In field observations of septic tanks having water softener brine discharged into the tank, the tanks have not developed distinct layers of sludge, scum, and a clear zone. These tanks were approximately four years old, and were expected to have a normal 3- to 4-inch thick sludge layer.

A study in Australia notes that “A loss of hydraulic conductivity results from the use of sodic wastewater on various soil profiles.” Nearly all ion exchange water softeners use sodium chloride for regeneration. The high concentration of sodium enters the wastewater stream when the water softener is backwashed. The sodium enters the wastewater stream as a slug to the septic tank two to three times per week.

In addition, field observations of side-by-side dispersal systems in a shared mound have shown that the trenches receiving the effluent with water softener brine discharges formed a thick, gelatinous slime layer that clogs the infiltrative surface, while the trenches receiving no salt water discharge remained open with a normal microbial clogging layer.

A study conducted at the University of Wisconsin introduced the backwash brine only to the soil dispersal component, not to the septic tank. The report suggests that additional research is needed to evaluate the effects of backwash brine on septic tank flora and fauna. The report is also inconclusive as to whether or not water softener backwash brine is harmful to septic systems. Additionally, these studies were performed nearly three decades ago when the required levels of treatment were not as restrictive as they are today.

The NSF study about water softener effects only compared one system receiving water-softener brine to a control system. The tests were performed to NSF Standard, 40 Class II requirements during 1978. These standards required that BOD₅ and TSS must not exceed 60 milligrams per liter (mg/L) and 100 mg/L respectively for more than 10 percent of the test period. Current requirements for most states are for NSF Standard, 40 Class I requirements of 25 mg/L BOD₅ and 30 mg/L TSS. Regulations require onsite treatment processes to accomplish not only higher organic removals, but also very restrictive nutrient removals. In these applications, water softener brine can be extremely detrimental.

**Sodium and Chloride Inhibit Operation**

Sodium concentrations greater than 3,500 mg/L have been reported to inhibit anaerobic digestion. It’s common to see municipal systems ban the discharge of high concentrations of salt into large treatment plants. Wastewater design texts and manuals require treatment processes to be sized accordingly, relative to influent salinity concentrations. “The higher the concentration, the greater the size” is not typically taken into consideration in small onsite applications.

Chloride concentrations greater than 180 mg/L have an inhibitory effect upon nitrifying microorganisms (U.S. Environmental Protection Agency Publicly Owned Treatment Works manual on toxics and inhibitory thresholds). Chloride concentration in regenerate can reach into the 10,000 mg/L range, with sodium in the 6,000 mg/L range. A field study of 18 onsite wastewater treatment systems in Virginia clearly showed that nitrogen removal was inhibited in systems receiving water softener backwash brine.

The systems receiving backwash brine from water softeners had average chloride concentrations of 1,207 mg/L in the septic tank effluent with one system having a concentration of 10,900 mg/L.
Manufacturers Void Warranties
Most of the reputable manufacturers of wastewater treatment systems have clauses in their warranties voiding the warranties if water softener backwash brine is discharged to the treatment system. There is a serious risk involved in discharging water softener backwash brine to advanced treatment systems. The risk is in the form of voiding the warranty, not meeting required compliance levels, tripling O&M needs/costs, and diminishing the long-term system performance and life.

Some regulatory authorities classify the brine as salt-laden water, free of contaminants to be dispersed directly to a sump or infiltration chamber. In most cases, regeneration brine began as well water (groundwater), and it is still considered groundwater with a heavy addition of sodium chloride (table salt).

Other Discharge Alternatives Exist
It is not necessary to discharge the regenerate to the wastewater stream. It’s done purely to cut costs. Homeowners are often told that it will cost “thousands of dollars” to re-route backwash brine away from the septic tank because it will involve the disruption and destruction of concrete footings and floors.

In Virginia, five water softener backwash discharges were routed out of the wastewater system for less than $100 per home. With simple planning at the beginning of the plumbing from the home, a second small pipe from the backwash could bypass the septic tank and the water softener backwash brine could be discharged away from the treatment system. If the soil scientist is comfortable with the salt water discharging to the soil, the pipe could lead around the septic tank and treatment system to the distribution box or discharge basin where the salt water could be diluted in the soil along with the remainder of the treated wastewater stream. The second pipe for water softeners could be included as a requirement in on-site regulations. The expense of a second pipe is minimal, and installing it along with the house sewer would reduce the cost compared to a retrofit.

Simple, inexpensive options are available to homeowners and regulators to prevent septic tank and treatment system failure and to keep the system warranty in effect.

Homeowners should have appropriate information to make informed decisions regarding their homes and the long-term effects one process/product may have on another. If they believe the risk is negligible, and are willing to discharge the backwash brine into their wastewater systems at the risk of voiding their systems’ warranty and increasing operating and other associated costs, they certainly have that choice.

In closing, as homeowners we all love our water to be soft. However, we all love it when our water and wastewater systems function and coexist efficiently, cost effectively, and cause us little concern. We also know that avoiding a minor installation expense at the risk of elevating service needs or premature pumpouts, repairs, rehabilitation, field replacement, performance/compliance costs, etc., is not a gamble worth taking. This is especially true when we know we can have our soft water without contaminating the chemistry of our wastewater processes.

References


Mark Gross, a former professor of civil engineering at the University of Arkansas in Fayetteville, is now the training manager at Orenco Systems, Inc. He has a BS and MS in civil engineering and a Ph.D. in engineering. He has more than 20 years’ experience in the decentralized wastewater field as a teacher, researcher, and designer, and is a registered professional engineer in Arkansas, Tennessee, Mississippi, Missouri, and Virginia.

Terry Bounds is the executive vice president for Orenco Systems. He earned his degree in engineering from Oregon State University in 1974 and is licensed in civil, environmental, manufacturing, and structural engineering. His primary focuses are R&D, installation and monitoring, O&M, and training and education.

![Diagram showing water supply, backwash, and drain](image_url)

The backwash phase reverses the water's flow and flushes any accumulated dirt particles out of the tank and down the drain. Next, in the regeneration or recharge phase, the sodium-rich brine solution flows from the brine tank into and through the mineral tank. The brine washes the calcium and magnesium off the beads. In the final phase, the mineral tank is flushed of the excess brine, which now also holds the calcium and magnesium, and the solution is disposed of down the drain.
Comparison of a Commercially Available Chlorine and an Ultraviolet Disinfection Unit for Onsite Wastewater Systems

AUTHORS
Harold Leverenz, P.E., Jeannie Darby, Ph.D., P.E., and George Tchobanoglous, Ph.D., P.E.

ABSTRACT:
Disinfection systems for onsite wastewater systems are required by regulatory agencies for some applications; however, there is insufficient operation and maintenance data available to ensure reliable performance. A commercially available calcium hypochlorite tablet chlorination unit and an ultraviolet (UV) disinfection unit were evaluated, under conditions within the operational range specified by the manufacturer, for suitability in onsite and small wastewater systems. The results are presented as a case study. The disinfection units were assessed based on overall performance, reliability and constraints, maintenance requirements, and estimated cost of installation and operation. Performance was evaluated by measurement of MS2 coliphage, total coliform, and fecal coliform inactivation. The disinfection systems were operated for nine months using biologically treated septic tank effluent. Both systems provided comparable results, frequently achieving 5 log removals. However, both systems were also subject to intermittent breakthrough events. Factors that were identified as important for effective performance included reliable pretreatment, flow equalization, and adequate maintenance.

Introduction
It is generally assumed that onsite wastewater systems that utilize subsurface dispersal achieve high levels of natural attenuation of pathogens through contact with the soil and its associated microbial community. However, there are numerous documented examples where groundwater quality has been compromised by pathogens derived from onsite wastewater systems, such as in areas where there is a high density of onsite treatment systems, cesspools, and seepage pits; shallow water table; or highly permeable soils (Ahmed et al., 2005; Nicosia et al., 2001; Arnade, 1999; DeBorde, 1998; Yates, 1985). While unsaturated flow conditions in soil dispersal systems can reduce indicator bacteria concentrations effectively, the transport of microbial pathogens from onsite wastewater systems is attributed to episodic breakthrough events (U.S. EPA, 2002).

Disinfection processes may be used with onsite wastewater systems when there is concern that there may be insufficient natural attenuation of pathogens in the soil, particularly in or near areas such as designated surface and groundwaters of exceptional quality, sensitive environments such as estuaries, adjacent to areas where shellfish harvesting is occurring, and sole source aquifers. Disinfection processes are also of interest when treated wastewater is to be distributed by surface spray irrigation or recycled for nonpotable domestic reuse applications (Asano et al., 2007).

Disinfection Methods
During the past 50 years, a wide variety of disinfection processes have been developed using physical, chemical, and biological agents (Asano et al., 2007). Disinfection processes that may be utilized for onsite wastewater systems are presented in Table 1. Of the disinfectants reported in Table 1, calcium hypochlorite and ultraviolet (UV) are used most commonly for small systems (U.S. EPA, 2002). Other processes identified in Table 1 that may be applied for wastewater disinfection include biological filtration (Gross and Jones, 1999; Vanlandingham and Gross, 1998; Emerick et al., 1997) and peracetic acid (Kitis, 2004). All of the processes identified in Table 1 can be used to disinfect wastewater; however, each process has inherent constraints that may limit general application. Details of each of these technologies may be found in Asano et al. (2007), Tchobanoglous et al. (2003), U.S. EPA (2002), and Crites and Tchobanoglous (1998). Disinfection with chlorine gas and chlorine dioxide are not considered as these...
processes present hazards associated with storage, handling, and application for small facilities.

Reliability and Performance Data for Onsite Disinfection Systems

Small treatment systems are expected to operate reliably under a number of challenging conditions, including long periods of time between maintenance activities, lack of redundant systems, high variability in flow rate and constituent concentrations, and site specific factors. Unfortunately, little information is available in the peer-reviewed and archived literature that can be used to determine the reliability and maintenance intervals required to keep a process performing to a high standard. Accordingly, the U.S. EPA (2002) states that inadequate data are available to assess the performance and reliability of tablet chlorination and UV disinfection devices, the most commonly used onsite disinfection systems.

Based on the field data that are available, concerns have been raised about the effectiveness and reliability of onsite disinfection units. For example, in field studies, tablet chlorinators have been found to have effluent concentrations of fecal coliform exceeding 200 MPN/100 mL in 93 percent of samples and no residual chlorine in 68 percent of samples (U.S. EPA, 2002). In another field study of 22 disinfection units used in conjunction with aerated biological treatment systems, it was found that UV systems were subject to lamp failure, and both chlorination and UV systems were subject to internal blockage and overflow, despite quarterly maintenance (Charles et al., 2003). The performance of the UV systems was dependent on the presence of particulate matter in the effluent from the pretreatment system, with some systems not able to meet the fecal coliform requirement of 100 CFU/100 mL. Charles et al. (2003) suggested that UV disinfection may be preferred over chlorination because the chlorination units produced high concentration of disinfection byproducts and poor reduction of indicator virus.

Experimental Methods

Two disinfection units for onsite wastewater systems were selected for evaluation, a calcium hypochlorite tablet feeder and a UV irradiation unit. The systems that were selected consisted of commercially available systems specifically marketed for use with onsite wastewater systems and are representative of the technology that is available currently and being utilized. Data provided by the manufacturer regarding the application of the disinfection unit, energy usage, and recommended service frequency are presented in Table 2. A description of the configuration of the disinfection systems, pretreatment system, water quality, and analytical methods used are presented below.

<table>
<thead>
<tr>
<th>Disinfectant</th>
<th>Formula</th>
<th>Form</th>
<th>Constraints or concerns for application to small flows</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sodium hypochlorite</td>
<td>NaCl</td>
<td>Liquid</td>
<td>Conserves, toxic, formation of carcinogenic by-products, requires chemical feed system, effectiveness may depend on water quality</td>
</tr>
<tr>
<td>Calcium hypochlorite</td>
<td>Ca(OCl)₂</td>
<td>Solid tablet</td>
<td>Conserves, toxic, formation of carcinogenic by-products, requires chemical feed system, effectiveness may depend on water quality, non-uniform tablet erosion may affect dose</td>
</tr>
<tr>
<td>Ozone</td>
<td>O₃</td>
<td>Gas</td>
<td>Conserves, toxic, requires a field gas preparation unit and a pump for injection of ozone, effectiveness may depend on water quality</td>
</tr>
<tr>
<td>Peracetic acid</td>
<td>CH₃CO₂H</td>
<td>Liquid</td>
<td>Conserves, toxic, not commercially available, requires a chemical feed system, effectiveness may depend on water quality</td>
</tr>
<tr>
<td>Ultraviolet (UV) light</td>
<td>-</td>
<td>UV radiation</td>
<td>Requires periodic lamp maintenance and replacement, fouling can reduce effectiveness, performance sensitive to water quality</td>
</tr>
<tr>
<td>Biological filtration</td>
<td>-</td>
<td>Enzymatic activity, precipitation</td>
<td>Size of filter may be a limitation, difficulty and expense of obtaining appropriate media</td>
</tr>
</tbody>
</table>

### Table 1

Summary of Disinfectants Used in Onsite Wastewater Systems
Wastewater Pretreatment Methods

The experimental facilities were located at the University of California, Davis, wastewater treatment plant. A portion of the raw wastewater from the UC Davis campus was diverted into a septic tank for primary treatment and then distributed to subsequent processes for secondary treatment. For the disinfection study, the septic tank effluent was treated using synthetic media biofiltration, also known as packed bed filtration (Leverenz et al., 2001). Packed bed filtration, including intermittently dosed sand and high-porosity synthetic media, is a biological process used in onsite and decentralized treatment applications because of its performance and reliability. The experimental configuration used for the testing of the disinfection units is shown in Figure 1.

Sampling and Analytical Methods

Water quality parameters, including total coliform, fecal coliform, MS2 coliphage, BOD$_5$, TSS, and turbidity, were measured in the laboratory within two hours of obtaining samples. Temperature and pH were measured at the research site when the samples were obtained. Total and free chlorine were measured 10 minutes (contact time) after taking samples. Undisinfected and disinfected effluent grab samples were obtained approximately once per week in sterilized Pyrex containers. Measurements for BOD$_5$ and TSS were conducted in accordance with Standard Methods (1998). Turbidity was measured with a Model #2100A Turbidimeter from HACH (Loveland, CO). Temperature and pH were measured using a combination handheld Ultrameter 6P from Myron L (Carlsbad, California). Total and free chlorine were measured using the DPD method with reagents and photometer from HF Scientific (Fort Meyers, Florida). Total and fecal coliform were enumerated using the membrane filtration technique and MS2 coliphage was enumerated using an agar overlay plating technique with E. coli #15597 as a host (Standard Methods, 1998).

Influent to Disinfection Systems

During the first 13 weeks of operation of the disinfection systems, the biological filters were operated to obtain an influent to the disinfection systems with BOD and TSS less than 30 mg/L. These performance criteria are the standard used for secondary treatment processes by the U.S. EPA and also represent the expected effluent quality resulting from many aeration processes.

Table 2

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Tablet chlorinator</th>
<th>UV</th>
</tr>
</thead>
<tbody>
<tr>
<td>Disinfectant</td>
<td>Calcium hypochlorite</td>
<td>Ultraviolet radiation</td>
</tr>
<tr>
<td>Manufacturer suggested application</td>
<td>Disinfection of water at flow rates to 5000 gal/d and untreated or treated septic tank effluent at flow rates to 500 gal/d</td>
<td>Disinfection of wastewater effluent with BOD and TSS &lt; 30 mg/L at flow rates up to 3 gal/min</td>
</tr>
<tr>
<td>Energy usage of disinfection unit</td>
<td>None</td>
<td>35 Watts or 306.6 kWh/y if operated continuously</td>
</tr>
<tr>
<td>Manufacturer recommended service frequency</td>
<td>Inspection and replacement of chlorine tablets on a semi-annual basis</td>
<td>Inspection every six months and lamp replacement every 2 years</td>
</tr>
</tbody>
</table>

Figure 1

Schematic Diagram of the Disinfection Study Configuration—Treated septic tank effluent was processed using a (a) chlorine tablet feeder and (b) UV disinfection unit. Note indicates sampling points.
For the following 24 weeks, the biological filter performance was optimized to obtain effluent BOD and TSS less than 5 mg/L, to simulate effluent that would be obtained from an intermittent packed bed filter or similar process. Details on the water quality used for testing of the disinfection systems during the study are summarized in Table 3.

Effluent from the biological filtration modules was collected in a common basin. A timer and normally-closed float switch were used to activate a pump in the collection basin and automatically fill a batch mix tank for the UV and chlorination systems on demand (see Figure 1). The timer was set to allow the pump to operate and fill the batch mix tank during a three-hour period in the morning and a four-hour period in the evening. When the batch mix tank was filled, the filter effluent would flow by gravity to the UV and chlorine disinfection systems.

The flow to each disinfection system was controlled with a gate valve, which was recalibrated twice per week to 1 gal/min. It was observed that the actual flow would fluctuate during the unattended loading periods, and ranged from 0.25 to 1 gal/min. The estimated daily loading to each disinfection unit was 187 gal/day.

Wastewater flow to the disinfection systems was stopped on 2/3/2005 and resumed on 2/26/2005 to simulate a vacation stress period. During the period of nonoperation, the UV lamp remained on while there was no flow through the unit. Before sampling, the flow rate to each disinfection system was set to 1 gal/min. Additional details on the evaluation procedure are presented in the following sections.

**Description and Configuration of Disinfection Systems for Testing**

Each of the disinfection units was configured according to manufacturer guidelines, while additional accommodations were made for flow rate control and sampling. Difficulties that were encountered with setup, operation, and maintenance are described for the relevant system. For each process, treated septic tank effluent was collected in a batch mix tank and inoculated with MS2 coliphage to obtain a high concentration (>10^9 phage/mL), as shown in Figure 1. A flow-control valve was used to obtain a flow rate of 1 gal/min. After a sufficient time to ensure that the system was at equilibrium (at least three system volumes flushed through), an influent and effluent sample were withdrawn and analyzed.

**Chlorine Tablet Feeder**

The dry chlorine tablet feeder, consisting of a flow-through chamber fitted with a single tablet feed tube, is designed for long-term operation with minimal maintenance. The calcium hypochlorite tablets (70 percent available chlorine) are designed to dissolve slowly as water flows through the tablet contact chamber. The tablet feeder is typically installed inline, but could also be used to treat a side stream that would then be blended with the bulk flow. The manufacturer states that the unit is acceptable for disinfecting septic tank effluent, effluent from a secondary treatment process, or drinking water.

There can be large variability in the chlorine dose required for untreated wastewater and drinking water; however there is little capacity for dose control using this type of chlorination system. The tablet feeder is typically installed subsurface, for example, in the effluent line of a septic tank. A schematic diagram and images of the chlorination system are shown in Figure 2.

**UV Irradiation Unit**

The UV irradiation unit is an assembly consisting of a tubular ABS pipe reactor with an axial germicidal lamp. The lamp housing is divided along the axis of the lamp such that water entering at the top of the unit flows downward past one side of the lamp, around the bottom of the lamp, and then upwards across the other side of the lamp before exiting at the outlet. The lamp is protected by a quartz sleeve, which is enveloped with a Teflon liner to inhibit surface fouling. The UV unit is marketed for onsite and decentralized wastewater treatment systems and is typically installed following a sec-

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Unit</th>
<th>Mean</th>
<th>Range</th>
<th>Mean</th>
<th>Range</th>
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</thead>
<tbody>
<tr>
<td>BOD₅</td>
<td>mg/L</td>
<td>14.3</td>
<td>6.7–91.0</td>
<td>1.2</td>
<td>0.8–1.9</td>
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<tr>
<td>TSS</td>
<td>mg/L</td>
<td>17.8</td>
<td>3.2–24.9</td>
<td>2.2</td>
<td>0.5–4.8</td>
</tr>
<tr>
<td>Turbidity</td>
<td>NTU</td>
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<td>7.4–33.6</td>
<td>2.2</td>
<td>0.5–2.9</td>
</tr>
<tr>
<td>pH</td>
<td>unitless</td>
<td>7.5</td>
<td>7.29–7.97</td>
<td>7.9</td>
<td>7.10–7.91</td>
</tr>
<tr>
<td>Temperature</td>
<td>°C</td>
<td>14</td>
<td>10.5–18.8</td>
<td>26</td>
<td>15.0–28.0</td>
</tr>
<tr>
<td>Total coliform</td>
<td>CFU/100 mL</td>
<td>1.9E9</td>
<td>7.4E4–3.8E6</td>
<td>1.7E5</td>
<td>4.7E4–6.5E5</td>
</tr>
<tr>
<td>Fecal coliform</td>
<td>CFU/100 mL</td>
<td>5.6E6</td>
<td>8.3E4–1.2E6</td>
<td>8.4E4</td>
<td>2.4E4–1.5E5</td>
</tr>
<tr>
<td>MS2 coliphage³</td>
<td>PFU/mL</td>
<td>1.8E6</td>
<td>8.6E4–3.8E5</td>
<td>5.0E5</td>
<td>1.6E5–2.3E5</td>
</tr>
</tbody>
</table>

³ Seed into influent of disinfection systems
ondary treatment device. The maximum flow rate recommended by the manufacturer is 3 gal/min. Additional units may be added in series or parallel to accommodate higher flow rates or higher UV dosages. The recommended influent water quality for both BOD and TSS is less than 30 mg/L, while a maximum turbidity value is not specified. A schematic diagram and images of the UV system are shown in Figure 3.

Results and Discussion
The results obtained for each disinfection unit are presented below. Performance measurements, reliability and constraints, maintenance requirements, and estimated cost of implementation and operation are discussed.

Chlorine Tablet Feeder
The overall performance of the tablet chlorination unit was excellent in terms of indicator bacteria and virus removal, with intermittent breakthrough events related to low residual chlorine dose. The low chlorine dose events were a result of differential tablet erosion, as discussed below.

Performance
The performance of the tablet chlorinator, evaluated at a flow rate of one gal/min., is presented in Figure 4. Total and free chlorine were measured following a 10-minute contact time. Sodium bisulfite (NaHSO₃) was added at this time (10 minutes) to stop the chlorine oxidation reactions. The free chlorine dose ranged from 0.14 mg/L to 390 mg/L (see Figure 4a). As shown in Figures 4b, 4c, and 4d, about 90 percent of the sampling events resulted in no coliphage or coliform organisms detected in the chlorinated effluent. There were three separate events where organisms were detected in the effluent; for these three events the corresponding free chlorine concentration was less than 1 mg/L.

It is interesting to note that the effectiveness of the chlorination unit was not affected by changes in effluent quality under the test conditions. The performance results are summarized in Table 4. While the manufacturer did not specify a contact chamber design, the 10-minute contact time resulted in high rates of disinfection due to the high free chlorine con-

![Figure 2](image2.jpg)

**Figure 2** Chlorine Tablet Feeder Used for Disinfection in Study—(a) diagram of tablet feeder, (b) image of unit installed for testing purposes, and (c) with tablet holder removed showing non-uniform dissolution of tablets.

![Figure 3](image3.jpg)

**Figure 3** UV System Used for Disinfection Study—(a) diagram of unit, (b) image of unit as installed for testing purposes, and (c) with fouled Teflon sleeve on left, UV lamp in center, and new Teflon sleeve on right.
centrations present. However, a contact basin with residence time greater than 10 minutes may provide better performance by moderating the variability in the chlorine concentration. A longer contact time may also be required if high concentrations of organic matter or ammonia, constituents which react with chlorine, are present in the effluent.

**Reliability and Constraints**

The primary issues related to reliability for the tablet chlorinator are (1) the dissolution rate of the hypochlorite tablets and (2) the ability to consistently apply a chlorine dose sufficient to cause disinfection. A summary of findings related to chlorine tablet usage and dose are presented in Figure 5. As shown in Figure 5a, periodic events with flow rates up to 2 gal/min. resulted in tablet decay at a rate of 0.4 tablet/day, while at a maximum flow rate of 1 gal/min., the decay rate was about 0.08 tablet/day. Therefore, without flow equalization, it would be difficult to predict the rate of tablet decay. Further, the manufacturer installation manual suggested the use of adapters for inline plumbing; however, the suggested installation caused the tablets to be submerged and to dissolve rapidly.

The results of several consecutive measurements made to characterize the chlorine dose as related to flow rate are shown in Fig. 5b. The chlorine dose applied was related to flow rate, duration and variability in flow rate, and age and condition of tablets. When new tablets were used, the free chlorine dose gradually decreased from 46 mg/L to 13 mg/L after 90 minutes of continuous operation. An identical chlorine dose test using tablets that had been in use for three months resulted in a higher initial free chlorine concentration (311 mg/L), followed by a rapid decline to 3.8 mg/L.

**Figure 4** Performance of the Chlorinator Unit—(a) chlorine concentration, (b) MS2 coliphage removal, (c) total coliform removal, and (d) fecal coliform removal.

**Table 4** Summary of Performance Characteristics of Tablet Chlorination Disinfection System

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Unit</th>
<th>Parameter</th>
<th>Parameter</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>MS2</td>
<td>PFU/mL</td>
<td>Maximum</td>
<td>164,333</td>
</tr>
<tr>
<td>coliphage</td>
<td></td>
<td>Minimum</td>
<td>98,514</td>
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<tr>
<td></td>
<td></td>
<td>Log reduction&lt;sup&gt;d&lt;/sup&gt;</td>
<td>0.8</td>
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<tr>
<td></td>
<td></td>
<td></td>
<td>&gt;6.4</td>
</tr>
<tr>
<td>Total</td>
<td>CFU/100 mL</td>
<td>Maximum</td>
<td>21,700</td>
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<tr>
<td>coliform</td>
<td></td>
<td>Minimum</td>
<td>12,400</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Log reduction</td>
<td>1.4</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>&gt;6.8</td>
</tr>
<tr>
<td>Fecal</td>
<td>CFU/100 mL</td>
<td>Maximum</td>
<td>1,870</td>
</tr>
<tr>
<td>coliform</td>
<td></td>
<td>Minimum</td>
<td>853</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Log reduction&lt;sup&gt;c&lt;/sup&gt;</td>
<td>2.1</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>&gt;5.2</td>
</tr>
</tbody>
</table>

<sup>a</sup> All values reported after 10-minute contact time

<sup>b</sup> Events with free chlorine concentration less than 1 mg/L are estimated to have occurred 10 percent of the time during the experiment. Organisms were only detected in the effluent when the free chlorine was less than 1 mg/L.

<sup>c</sup> Log reduction = -log (effluent concentration / influent concentration)

<sup>d</sup> Reported as mean log reduction
after 60 minutes of continuous operation. Although water only contacts the bottom tablet, it was observed that several tablets located above those also began to dissolve. The dissolution of the elevated tablets was caused by condensation and may have contributed to some of the extreme measurements. It was also found that the chlorine tablets did not dissolve uniformly, and sometimes only a small amount of the chlorine tablet was actually in contact with the water, as the bottom tablets had eroded and formed a channel where water could pass through with little contact. On other occasions, large amounts of particulate calcium hypochlorite were washed out of the reactor with the effluent. Stopping and restarting of flow caused chlorine concentration peaks followed by a rapid decline (data not shown).

A summary of all chlorine concentrations measured while obtaining samples is shown in Figure 5c using a probability distribution. As shown, the average concentrations measured were 11 and 20 mg/L, for free and total chlorine, respectively. The free chlorine concentration exceeded 200 mg/L 10 percent of the time and was less than 1 mg/L 10 percent of the time.

The chlorination unit was not affected by the three-week vacation period (i.e., no flow conditions). In addition, the chlorination performance was not affected by variations in water quality that occurred during the study. However, the effect of high residual chlorine in the effluent on the ability of the soil bacteria to provide advanced treatment is not known. Therefore, dechlorination facilities (also tablet feed), following chlorine contact basins, may be considered to control the discharge of chlorine where there is concern about the receiving environment.

**Maintenance Requirements and Frequency**

Maintenance for the tablet chlorination unit consisted of periodically refilling the tablet feed tube with new tablets and checking the condition of the existing tablets. Assuming flow equalization and uniform tablet dissolution, it is estimated that the unit could be operated for several months without servicing to replace tablets. However, Weaver and Lesikar (2004) recommended that only two to five tablets be used to reduce failure due to compaction and non-uniform tablet erosion. The current research also supports the suggestion of Weaver and Lesikar (2004) that chlorine tablet feeders should be checked weekly to ensure that tablets are present and feeding properly. Although optimal maintenance is frequent, the ease of that maintenance is an advantage for this technology.

**Estimated Cost of Installation and Operation**

The expenses associated with installation, operation and maintenance are presented in Table 5. The capital cost for the tablet chlorinator is $150
and the cost of a year’s supply of calcium hypochlorite tablets (a 10-lb supply, about 30 tablets) with shipping is approximately $100. Additional expenses for installation of flow equalization, contacting facilities, and dechlorination facilities result in an overall estimated capital and installation cost around $8950. The annual operation and maintenance costs for the tablet chlorination unit are estimated to be around $1,050.

A maintenance interval could not be established based on this research because of the unpredictable nature of the tablet chlorination device. Therefore, a conservative maintenance interval ranging from weekly to twice monthly service is recommended, depending on the importance of reliability and performance of the disinfection system. The draft onsite wastewater regulations for the state of California require weekly maintenance of disinfection facilities, unless telemetric monitoring is used (State of California, 2007). Remote monitoring systems for residual chlorine concentration are available but would probably not reduce the maintenance interval substantially due to the need to replenish reagents and calibrate the equipment. The cost for maintenance will also depend on availability of a service provider and area-wide monitoring program.

**Ultraviolet Unit**

With regular servicing, the UV unit was effective for the reduction of indicator organisms. Hardness minerals present in the potable water supply precipitated on the lamp sleeve and reduced the performance of the UV unit over time. Cleaning of the lamp sleeve was required to maintain the high level of disinfection capacity, as discussed below.

**Performance**

The performance of the UV unit for inactivation of MS2 coliphage, total coliform, and fecal coliform is shown in Figure 6 and summarized in Table 6. The performance was affected by the quality of effluent from the pretreatment system, with best performance occurring when the pretreatment process was producing effluent with TSS less than 5 mg/L and turbidity less than 3 NTU. The three-week period with no flow (2/3/05 to 2/26/05) severely impacted performance, reducing removal rates for MS2 and coliform bacteria.

After several sampling events between 2/26/05 and 3/11/05, the UV unit was inspected to determine the cause of the reduced performance. It was found that water had entered into the space between the quartz sleeve and Teflon lining. According to the manufacturer, water was not supposed to enter the space inside of the Teflon lining; therefore, the entry occurred due to a manufacturing or handling defect.

Two types of surface fouling were also identified during the 3/11/05 inspection. The first type of surface fouling was present on the outside of the Teflon lining and consisted of a powdery white substance, concluded to be precipitated calcium carbonate hardness minerals. The precipitate evenly covered the entire length of the Teflon liner in all areas that were submerged and exposed to UV light. This white precipitate coating was easily removed by wiping the Teflon lining with a towel. The second type of fouling was present on the inside of the Teflon lining and was brown in color, covering about 25 percent of the area exposed to UV. It was concluded that the brown discoloration was due to a reaction between humic substances in the water, increased water temperature, and the UV radiation. It was observed that both instances of fouling were only present in areas on the liner that were submerged and exposed to UV radiation, and not present in areas that were out of direct exposure, e.g., the area around the cap on the end of the lamp. The brown surface fouling on the interior of the Teflon lining was due to the water intrusion, but the time and manner of water intrusion was not known. It was not possible to remove the brown fouling in the field, thus a new lamp housing assembly was acquired and installed on 3/16/05.

On 3/16/05, the performance of the pretreatment systems was optimized to improve the influent water quality to the disinfection systems (see Table 3). As shown in Figure 6, the performance of the UV unit improved following the optimization of the pretreatment system and lamp maintenance events. The average removal of MS2 coliphage after optimization of the pretreatment system was 5 log (99.999 percent) during the testing with high quality effluent at 1 gal/min. Using data on MS2 inactivation from U.S. EPA (2003), the estimated applied UV dose was 105 mJ/cm².

### Table 5

<table>
<thead>
<tr>
<th>Category</th>
<th>Item</th>
<th>Estimated cost, US$</th>
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</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Tablet chlorination</td>
</tr>
<tr>
<td><strong>Capital costs</strong></td>
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<tr>
<td>Equipment</td>
<td>Equalization tank</td>
<td>150</td>
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<tr>
<td></td>
<td>Tablet reader (2)</td>
<td>600</td>
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<tr>
<td></td>
<td>UV unit with lamp assembly</td>
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</tr>
<tr>
<td></td>
<td>Contact tank (2)</td>
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<tr>
<td></td>
<td>Access container</td>
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<tr>
<td>Installation</td>
<td>Excavation and backfilling</td>
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<td>Electrical work (trenching,</td>
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<td>conduit, wiring)</td>
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<td></td>
<td>Piping and fittings</td>
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<td><strong>Subtotal - capital</strong></td>
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<td>850</td>
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<td><strong>Operation and maintenance costs</strong></td>
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<td><strong>(annual)</strong></td>
<td>Site visits (twice monthly)</td>
<td>600</td>
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<tr>
<td></td>
<td>Chlorination</td>
<td>120</td>
</tr>
<tr>
<td></td>
<td>Dechlorination</td>
<td>150</td>
</tr>
<tr>
<td></td>
<td>UV lamp (replaced every two years)</td>
<td>40</td>
</tr>
<tr>
<td></td>
<td>Electrically</td>
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<tr>
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<tr>
<td></td>
<td>(quarterly)</td>
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<tr>
<td><strong>Subtotal - O&amp;M</strong></td>
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</table>
There was a gradual increase in the effluent MS2 coliphage concentration following the first lamp maintenance event. The lamp assembly was removed on 6/6/05, 75 days after the first lamp maintenance. The carbonate precipitate was present on the Teflon lining of the UV unit. In addition, the Teflon liner was once again found to have a small tear and water inside of the liner; however, the brown fouling was not present. It was concluded that the water intrusion was not inhibiting the effectiveness of the unit. The precipitate was removed from the Teflon liner and the lamp assembly was replaced and flow restarted. Cleaning the lamp had a positive effect on the performance of the UV unit. Without frequent measurement of indicator organisms, it would not have been possible to determine the effectiveness of disinfection. While the precipitate was clearly visible by direct observation, the relationship between the observed fouling and disinfection performance is not known. Some UV units are equipped with a sensor to monitor UV output and may be used to alert a user that the lamp has failed or the performance has been compromised and initiate lamp maintenance or lamp replacement.

During normal operation of the UV unit it was noted that suspended solids accumulated in the bottom of the reactor during extended periods of low flow. Increases in the flow rate through the reactor caused these solids to be flushed out of the system with the effluent. The solids accumulation and flushing was most noticeable when the influent TSS was elevated, but solids resulting from growth and detachment of biofilm within system piping may also contribute to solids loading.

![Figure 6](image)

**Figure 6**

Performance of the UV Unit—(a) MS2 coliphage removal, (b) total coliphage removal, and (c) fecal coliform removal. The vertical dashed line indicates removal and cleaning of precipitates from the lamp housing.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Units</th>
<th>Parameter</th>
<th>Value</th>
<th>Value</th>
<th>Value</th>
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<td>MS2</td>
<td>PFL/mL</td>
<td>Maximum</td>
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<td>264</td>
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<tr>
<td></td>
<td></td>
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<td></td>
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<td>&gt;5.0</td>
</tr>
<tr>
<td>Total</td>
<td>CFU/100 mL</td>
<td>Maximum</td>
<td>117</td>
<td>13,193</td>
<td>240</td>
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<tr>
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<td></td>
<td>Mean</td>
<td>46</td>
<td>7,522</td>
<td>24</td>
</tr>
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<td></td>
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<td>Log reduction</td>
<td>4.0</td>
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<td>&gt;4.8</td>
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<tr>
<td>Fecal</td>
<td>CFU/100 mL</td>
<td>Maximum</td>
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<td>1,200</td>
<td>13</td>
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<td></td>
<td></td>
<td>Mean</td>
<td>0</td>
<td>737</td>
<td>1</td>
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<td></td>
<td></td>
<td>Log reduction</td>
<td>4.3</td>
<td>3.1</td>
<td>&gt;4.4</td>
</tr>
</tbody>
</table>

Table 6

Summary of Performance Characteristics of the UV Disinfection System.

---

- **Parameter**: Measurement or condition being evaluated.
- **Units**: Units of measurement.
- **Performance value**: The value of the parameter, with units specified.
- **during given time period**: The time period during which the data was collected.

- **12/16/04**: Data collected on 12/16/04.
- **2/26/06**: Data collected on 2/26/06.
- **3/22/06**: Data collected on 3/22/06.
- **3/21/06**: Data collected on 3/21/06.

---

- **Loading was discontinued on 2/26/06 and resumed on 3/22/06; lamp removed on during the entire period of flow**
- **Severe fouling occurred during the 20 day without flow, lamp maintenance occurred on 3/21/06**
- **Lamp maintenance occurred on 6/6/06**
- **Log reduction = log( (effluent concentration / influent concentration) )**
- **Reported as mean log reduction**
The effect of the solids flushing is expected to reduce the performance by shielding organisms and possibly providing a habitat for biological growth within the reactor. While performance measurements were not made during the solids flushing events, a pretreatment process capable of producing effluent with low TSS is considered to be an important factor in UV disinfection performance. Up-flow and horizontal flow reactor configurations may not be as subject to the particular accumulation issues.

**Reliability and Constraints**

Several constraints were identified that affected the reliability of the UV unit. The constraints were the mineral characteristics of the water supply used, the influent water quality from the pretreatment systems, and the period while the lamp was left on without flow.

Under optimum conditions, the UV unit was able to reduce MS2 coliphage and coliform bacteria concentration effectively. The manufacturer’s recommended inspection interval is every six months. However, based on the findings of this research, maintenance may be required more frequently due to the influent water quality and the level of disinfection desired. The hardness present in the water supply used for testing was implicated in the increased maintenance needs. Therefore, the specified maintenance interval should be based on water quality parameters and disinfection requirements.

The type and performance of the pretreatment system needs to be taken into consideration with respect to the effectiveness of the UV unit. When operated with a lower water quality, performance of the UV unit was reduced and additional lamp fouling may have occurred from the presence of increased organic matter present in the water. The accumulation of solids in the bottom of the reactor was also a consequence of using water with moderate levels of residual TSS. Therefore, the type and reliability of the pretreatment system are important factors for the implementation of UV disinfection.

The flow variability from onsite and decentralized treatment systems may have a negative impact on UV type disinfection systems. For example, the process would be subjected to both periods of high flow and no flow. At high flow rate events the UV unit may not provide an adequate dose for effective disinfection, while the no flow condition will result in the stagnant water being heated by the lamp, resulting in increased precipitation of some water constituents (if present). Therefore, flow equalization and water quality should both be considered for implementation of UV disinfection. Recirculation of flow through the UV unit may help to reduce the temperature increase while simultaneously increasing the contact time and applied UV dose.

**Maintenance Requirements and Frequency**

Maintenance of the UV unit consisted of (1) stopping flow through the unit, (2) disconnecting the power supply, (3) removal of the lamp assembly, and (4) cleaning of the Teflon liner. The precipitate was removed easily from the Teflon liner using a cloth, whereas an unprotected quartz sleeve would have required the use of acidic chemicals. In addition, any solids deposited in the bottom of the reactor should be removed by flushing with water.

During this study, punctures were found in the Teflon liner that allowed water to come into contact with the quartz sleeve. The intrusion of high quality water did not adversely impact performance of the unit, while intrusion of water with partial treatment caused fouling inside of the Teflon liner. Therefore, the Teflon liner, if present, should be inspected carefully for punctures and replaced if necessary, particularly if there is doubt about the performance of the pretreatment system.

The maintenance frequency depends on several factors, including (1) the organisms to be inactivated, (2) the required degree of inactivation, (3) background water quality related to potential for precipitation of minerals, and (4) reliability and performance of the pretreatment system. For this study, it was found that the UV unit could operate for about 30 days after lamp maintenance without detection of MS2 in the effluent, and 75 days before detection of coliform bacteria. However, if the permissible concentrations of indicator organisms are higher than none-detected, extended periods of operation may be acceptable. Determination of maintenance intervals should therefore be determined according to the factors cited above and the capacity for natural attenuation in the receiving environment.

**Estimated Cost of Installation and Operation**

The estimated capital, operation, and maintenance costs for the UV unit are summarized in Table 5. The overall capital and installation cost for the UV unit is estimated to be $1,500. The estimated operation and maintenance costs are around $890, which includes the cost of replacing the lamp every two years (manufacturer recommendation) and 300 kWh of annual energy usage.

Based on the findings of this research, a maintenance interval of about two weeks would ensure control of coliphage virus, although human pathogens may respond differently to the applied UV dose. For comparison, the draft onsite wastewater regulations for the state of California require weekly maintenance of disinfection facilities, unless telemetric monitoring is used (State of California, 2007). As with the chlorine unit, the actual maintenance interval prescribed should be based on the importance of reliable performance for the site under consideration. UV transmittance sensors may be used for remote monitoring of the UV system, which will affect the maintenance cost. As for chlorine, the cost for maintenance will also depend on availability of a service provider and area-wide monitoring program.

**Conclusions**

- The operation and maintenance required for high levels of disinfection performance will depend on site-specific factors, including source water quality, constituents added during water use, application and performance of the upstream facilities, and installation characteristics of the disinfection system, and provision of flow equalization. However, with proper attention, both the UV and chlorination systems evaluated can be made to operate reliably.
• Proper installation and monitoring was necessary to ensure that the disinfection systems were operating properly. The manufacturer recommended maintenance schedules were not sufficient to achieve the highest level of performance. The minimum recommended frequency of maintenance for disinfection technologies evaluated under the conditions of this study is twice per month.

• The systems tested in this study failed in a number of ways that would not have been apparent without monitoring for indicator organisms. The tablet chlorination system was susceptible to episodic failure due to non-uniform erosion of tablets, while the UV system was subject to progressive failure as fouling occurred on the lamp housing (e.g., Teflon liner).

• The effect of erosion of calcium hypochlorite tablets on the chlorine dose is difficult to predict and is not related to water quality or chlorine demand. The rate of tablet erosion can be variable if flow equalization is not used or if several tablets are added simultaneously.

• To minimize the impact of residual chlorine on soil microorganisms and receiving surface and subsurface waters, dechlorination may be considered for chlorination devices. The generation and effects of disinfection byproducts in the soil are unknown when chlorination is used. Disinfection with UV does not generate any known byproducts at the UV dose applied.

• The UV system was sensitive to water mineral content, periods of no flow while the lamp remains on, and reliability of the pretreatment system to provide adequate water quality. The chlorination system was not as sensitive to these factors.

• All disinfection systems should be used in conjunction with some type of flow equalization to minimize the peak flows expected from small wastewater systems. Increased flow rate through the disinfection units tested resulted in excessive tablet erosion rates and reduced UV dosages. Flow equalization may be provided as a stand-alone process or as part of the facilities upstream disinfection.

• The chlorination system for an individual residence is expected to have capital and annual operation and maintenance costs around $950 and $1,050, respectively. The UV unit is expected to have capital and annual operation and maintenance costs of $1,500 and $890, respectively. It is recognized that the cost for operation and maintenance from a service provider will be site specific. Monitoring of the UV system with a telemetry system may reduce the maintenance costs.

Acknowledgements
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References


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A Rational Method for Determining Design Flows for Cluster Systems

ABSTRACT:
Designers and regulators working with cluster wastewater treatment systems often face difficulties determining the appropriate design flow for housing developments due to a lack of information. Studies have attempted to determine water use or wastewater discharge per capita, and these numbers have found general acceptance around the country, but there is not as much consensus regarding how these numbers should be applied to proposed housing developments in any given community. Factors such as income level, home size, and lifestyle are thought to influence household occupancy, and, therefore, water use. It is common practice to simply multiply the number of homes and bedrooms by the local prescriptive design number of gallons per bedroom used for onsite systems to estimate a total flow for a group of homes. This paper discusses factors that need to be considered in developing a basis of design for a cluster wastewater system and sets forth a methodology for using the demographics of the surrounding community, obtained from the latest census data, to estimate a design flow for any project. A comparison is made with actual design flows recorded for a number of residential cluster projects in Michigan.

Background
One concept in the design of sewer infrastructure for new developments (and sometimes for communities of existing homes) in Michigan and elsewhere in the U.S. is to provide one or more smaller collection and treatment systems for small groups or “clusters” of homes, rather than one large centralized system. This concept sometimes represents the most cost-effective method of wastewater management for the community.

Because these clusters of homes vary in size from a very few homes to dozens or more, they present some interesting challenges for the designer and regulatory community with regard to predicting the wastewater flow quantities. Smaller numbers of homes can be expected to exhibit larger flow variability than larger numbers of homes, where periods of peak usage from individual homes tend to mitigate one another. The goal of this paper is to set forth a rational method of predicting expected wastewater flow from clusters of homes of different sizes and to accompany that projection with statistical confidence.

Many studies have been performed and much has been written concerning what the per capita wastewater flow is from residential communities. The conclusions of many of these studies indicate per capita daily flows of 50 to 55 gallons per day (GPD)—some estimate as high as 60 GPD (McEachin and Loudon, 2002; EPA, 2002). There seems to be widespread agreement on the use of these numbers for design purposes, particularly for homes built after 1994 with water-efficient fixtures and appliances.

However, when it comes to estimating the population of a home or group of homes, there appears to be less agreement. Both the expected number of people living in a neighborhood and the flow per person are necessary parameters in estimating the total design flow, if you happen to be a decision-maker in the design of the wastewater system to serve a new community. Little has been written with regard to the proper sizing of systems serving smaller communities of homes. Is the size of the homes a critical factor? Is the number of bedrooms or bathrooms a factor?

Fact or Fiction?
Most codes for onsite wastewater treatment systems are written with prescriptive language that requires the system to be sized based upon the number of bedrooms in the dwelling. Two common flow formulas set forth in codes are:

- 150 GPD for the 1st bedroom + 100 GPD/each additional bedroom, or
- 150 GPD per bedroom

Do either of these formulas provide an accurate estimate of the actual flow? Is there any relationship between these numbers and household occupancy? How do these formulas relate to the actual flow from a group of homes?
Some would argue that these numbers should still be used to allow for peak flow conditions, because you can’t control the number of people living in a home. Those arguing the contrary point of view would say that the peak flow conditions are mitigated by larger numbers of homes, with actual flows coming closer to the averages. But what are the facts?

Census Information
The 2000 U.S. Census information is readily available online for any jurisdiction in the U.S. at [http://www.factfinder.census.gov/home/saff/main.html](http://www.factfinder.census.gov/home/saff/main.html) For any community, it is possible to obtain household size and population information. Following are some interesting census data facts for the state of Michigan:

- The average household size for Michigan is 2.56 people.
- Less than four percent of households have more than five people living in them.
- Less than 11 percent of households have more than four people living in them, even though more than 17 percent of households have more than three bedrooms.

So, if one were designing a central wastewater system for the entire population of the state of Michigan and used the numbers found in many codes, it would appear that it would be considerably over-designed! Similarly, it may be true that smaller systems for communities of homes are over-designed if these same code formulas are used.

Social and Economic Factors
Some would argue that social and economic factors have a large affect on the expected wastewater flow: for instance, the larger the home, the more people that are likely to reside there, and the more wastewater that is likely to be generated; or the more affluent the neighborhood, the larger the homes and the families likely to be living there. However, some might argue the opposite. How significant are these factors? Or, are these factors at all?

The author analyzed the 2000 census data for 12 communities in Michigan. The 12 communities were selected because they represented somewhat of a cross-section of the Michigan population and included small, large, lower income, wealthy, urban, and rural communities. The goal of the analysis was to determine whether household income and home size had any effect on the household occupancy.

Table 1 shows the results of this demographic comparison. Figures 1 through 4 graphically show the trends as median household income increases. The communities have been ordered from one to 12—from lowest to highest household income. As expected, the size of the homes and the number of bedrooms increases as household income increases. It is, however, somewhat of a surprise that the average number of bedrooms does not even reach four in the higher income communities. And, I think most would be surprised to know that the average household occupancy remains almost constant at around 2.5 people per household, regardless of income. In fact, the trend is slightly downward with increased income.

Therefore, since we know that people produce wastewater, not bedrooms or bathrooms, it would seem logical that we should make use of this readily available information in the sizing of our wastewater treatment works. Census data is

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**Table 1**

<table>
<thead>
<tr>
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<td>$170,790</td>
<td>9.0</td>
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Total Population: 1,475,816

(Avg.)
available for any local jurisdiction by going to the aforementioned Web site. With a series of steps to define your local census district, one can then obtain census findings on the percentage of homes with one occupant, two occupants, and up to seven occupants. By using these percentages to compute the expected population in any particular size community, and plugging in the flow per capita to be used, one can easily compute the design flow to be used for the treatment works. From there, the designer can use a safety factor of his or her choosing to allow for peak flow conditions.

Figure 5 is an example of a spreadsheet calculation using census information for a new housing project of 41 homes in Handy Township, Livingston County, Michigan. The upper section of the spreadsheet lists the 2000 census data downloaded from the aforementioned census information Web site for Handy Township in Michigan. Handy Township is a typical rural governmental jurisdiction of approximately 36 square miles. In this example, Handy Township has a total of 2,477 homes according to the 2000 census.

The lower section of the spreadsheet applies the percentage of homes with the various occupancy numbers to the total number of homes in the proposed development (41) to estimate the number of homes that will have 1, 2, or 3, etc., occupants. The spreadsheet uses these numbers to then estimate the anticipated population in the new development. For our example, the estimated population of the 41-home development is 113 people, or 2.76 people per household.

The last two lines of the spreadsheet simply allow the designer to then use a per-capita average and peak flow number of his or her choosing to calculate the average and peak design flow for the project.

**Flow Variability**

We must always keep in mind that illustrated here is a method of estimating the likely population of a new community; and, from that estimated population, determine an anticipated wastewater flow. It could also be used to estimate current flow from an existing community, but...
would not be as accurate as an actual population survey and/or actual flow measurement. This method assumes that the new development will have a population that mimics the characteristics of the larger surrounding community. Designers are cautioned to decide if that is a valid assumption for any particular project.

Certainly, there is variability in statistical data, so we must recognize the limits of our procedure. With regard to methods listed here, the following cautions are in order:

- The methods discussed here are appropriate for calculating the wastewater quantity generated at the source. If one is to use the same quantities for design deci-

![Figure 4](image-url) Average Household Size by Community

<table>
<thead>
<tr>
<th>Livingston County - Handy Township</th>
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<tr>
<td><strong>Household Size - 2000 Census</strong></td>
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<tr>
<td><strong>Number</strong></td>
</tr>
<tr>
<td>1-person household</td>
</tr>
<tr>
<td>2-person household</td>
</tr>
<tr>
<td>3-person household</td>
</tr>
<tr>
<td>4-person household</td>
</tr>
<tr>
<td>5-person household</td>
</tr>
<tr>
<td>6-person household</td>
</tr>
<tr>
<td>7-or-more person household</td>
</tr>
<tr>
<td><strong>Total Households</strong></td>
</tr>
</tbody>
</table>

**Applying This Information To Summerbrook**

<table>
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<tr>
<th>Using Handy Township Data</th>
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<tr>
<td><strong>Number</strong></td>
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<tr>
<td>1-person household</td>
</tr>
<tr>
<td>2-person household</td>
</tr>
<tr>
<td>3-person household</td>
</tr>
<tr>
<td>4-person household</td>
</tr>
<tr>
<td>5-person household</td>
</tr>
<tr>
<td>6-person household</td>
</tr>
<tr>
<td>7-or-more person household</td>
</tr>
<tr>
<td><strong>Total Households</strong></td>
</tr>
</tbody>
</table>

**Total Anticipated Population** = 115 People per household

**Total Anticipated Average Flow** = 115 X 60 GPD/Capita = 6900

**Total Anticipated Peak Flow** = 115 X 75 GPD/Capita = 8625

![Figure 5](image-url) Spreadsheet Used in Applying Census Data to a Project
Editions at the downstream end of the collection system, the collection system must be watertight. Any infiltration, inflow or exfiltration has not been accounted for.

- No attempt has been made in the methods suggested here to accommodate different lifestyles. As examples, recreational communities may have significantly different occupancy patterns; homeschooling of children may increase flows from homes; and significant hot tub and home spa use could result in increased volume.
- Some professionals express a legitimate concern that a characteristic of more affluent lifestyles is more frequent entertaining, resulting in higher wastewater flows. Designers need to be conscious of this factor as they choose safety factors for their system component sizing, particularly for systems serving smaller communities. Peak flows in portions of the system are mitigated to some extent as the size of the system grows. So, in general, the author suggests that designers use a larger factor of safety for systems serving smaller communities.

However, with regard to the census data, we can draw some statistical conclusions. As an illustration of how this would be done, the census data for household size was obtained for a typical township in southeast Michigan. It has a population of about 6,000 households. The census data obtained for this township was used to plot confidence intervals for different size populations using a standard spreadsheet program.

In this way, we can determine with 90 or 95 percent confidence what the expected population will be for any particular size community. Therefore, flow differences based upon population variability can be factored into the safety factor chosen for the design. Figures 6, 7, and 8 illustrate how this data can be useful in projecting a design flow per household, or for an entire community. For the purpose of this illustration, a per capita flow of 60 GPD was used.

Note that statistically this data becomes much less useful with small numbers of homes because
the sample size is too small to be statistically predictable. Notice from Figures 6 and 7 that as the number of homes approaches zero the confidence intervals widen off the chart. For instance, for a sample size of only one home, an accurate statistical prediction cannot be made. In such cases, the designer may get some design guidance from census information as to how many homes in that community have a household occupancy of, for example, five or more people, to help him or her quantify the risk for design purposes.

**Comparison With Actual Projects**

So, how does this method of predicting project flows stack up against actual field measurements? To answer this question, the actual flow records from 14 different residential projects located in southern Michigan were obtained. Each of the projects had a means of measuring the actual wastewater quantities being treated at the treatment site. The type of collection system varies—some with grinder pumps and pressure sewer, some with STEP systems, and some with gravity sewers. With two exceptions, the data available was only the total annual flow that could be averaged to estimate the average daily flow. Two of the projects were equipped with telemetry panels that logged daily flow information.

Table 2 lists the actual measured data from these 14 sites, ranging in size from 7 homes to 272 homes. The average daily flow from the total project is listed, together with a calculated average daily flow per home. The right side of Table 2 illustrates a comparison of the actual recorded flows with the flow that would have been predicted using the census methods described above. The population of the development is first estimated using the census data of the surrounding community. This number is multiplied by 60 GPD per capita (the flow chosen for this illustration) to obtain an estimated average daily flow for that size community. This table then shows the difference between the actual and the estimated flows. A red number in that column indicates the actual measured flow was larger than what the predicted average flow would have been based upon our assumed per capita flow and estimated population. A black number indicates our predicted flow was higher than the actual measured flow. Readers should keep in mind that this table does not illustrate confidence intervals, nor does it illustrate peaking factors. It is a simple comparison of predicted and measured averages.

Superimposed on Figure 8 are the data points from a full year of data from each of the 14 operating facilities. Some of the numbers from existing facilities are so close that the points actually overlap and hide one another at this scale. One of the projects was too large for this graph, so the information is listed below Figure 8.

As stated above, the projects listed had a variety of collection systems. The systems that used gravity collection, or otherwise were vulnerable to infiltration and inflow, would have measured flows that included the extra leakage. In addition, some of the systems utilized uncovered recirculating sand filters at the treatment works. These components do allow precipitation to enter the system prior to flow measuring devices, and the extra water would be recorded as flow. Without some investigative effort (considered beyond the scope of this paper), it is not possible to quantify the extra infiltration and inflow. Obviously, the designer must consider these extra sources of flow unless they are somehow prevented from entering the system by design.

As one compares the actual recorded flows with the statistically predicted flows shown in Figure 8, a casual observer could get the impression that the smaller systems generated flows closer to the average predicted numbers than the larger systems. The author suggests that system infiltration and inflow may be an explanation for this phenomenon. Remember, a per capita average daily flow of 60 gallons per person was used to generate Figures 6, 7, and 8. This number is on the high side of numbers found in the references, and seems to overestimate flow experienced in the larger systems mentioned here. But, proportionately, flows recorded for the smaller systems seem to meet or exceed this number. In other words, smaller systems seemed to produce more flow per capita than do the larger systems.

The author suggests that the observation can also be explained by the fact that infiltration and inflow may have a larger impact on recorded flow for the smaller systems than for the larger systems. This is supported by a more detailed analysis of the daily flow records of two of the systems that have recirculating sand filters. Brooks River Landing and River Rock Landing are projects located near Lansing, Michigan, for which the author has several years of daily flow records. Flow spikes are readily observable in these flow records during precipitation events.

As one can see, the actual data from all 14 of these existing communities falls within the expected flow range (using 60 GPD/capita) predicted by the methods described earlier. It would appear from this information that the predicted average flow could safely be used for projects of over 20 homes with a safety factor allowance of the designer's choice for peak days.

**Safety Factors**

Described above is the author's suggestion for estimating a design average flow for projects based upon probabilities. Choosing a safety factor for any particular component of the collection or treatment works should follow the same type of rational analysis. Designers should ask themselves one key question: What would happen if the flow through any particular component is greater than the design flow for that component? The follow-up analysis of that answer should take a rational path as follows:

- Those components that would cause a system “failure,” in the sense that the system would be incapable of performing within compliance limits, need to be sized substantially larger than the highest expected flow, leaving room for predictive error.
- Those components that would still perform, but would result in some reduced system performance under higher flow conditions, may or may not need to be up-sized. The designer needs to analyze how often and how long the high-
er flow conditions might occur, and balance that against the buffering ability of the system to handle such flow conditions. He or she also needs to consider the cost-effectiveness of enlarging the capacity of that component.

• Those components that would have negligible impact on overall system performance when experiencing occasional higher-than-expected flow conditions do not normally justify up-sizing for peak flow conditions, particularly if those higher flow conditions are rare occurrences.

Table 3 is an example of the ratio of measured peak flow compared with the average measured daily flow for one cluster system located near Lansing, Michigan. This is one of the facilities that the author has logged daily flow data over several years via telemetry at the treatment site. Shown in Table 3 are two full years of data from the third quarter of 2002 through the second quarter of 2004. Also shown is the highest daily flow measured during each quarter, and the ratio of that one-day high, to the quarterly average.

This system is comprised of a recirculating sand filter, followed by an intermittent sand filter. While the collection system in this housing development is watertight, any precipitation on the R.S.F. is captured in the system and included in the measured flow. A careful review of precipitation records shows a strong correlation between rainfall events and the peak flow occurrences. Designers are cautioned to consider such characteristics when interpreting and applying this information to other projects. One might note, however, that if 60 GPD per capita were used

| Table 2 | Comparison of Actual Measured Flows with Predicted Flows Using Census Data for the Surrounding Community |
|-----------------|-------------------------------------------------|-------------------------------------------------|-------------------------------------------------|-------------------------------------------------|-------------------------------------------------|
| Site Information | Measured — 2003 | Flow Estimate Using Census Data 60 GPD/Cap | Difference from Predicted @ 60 GPD/Cap | Est. Population | Est. of Meas. Flow per capita |
| Sewer District | Type | Initiated | No. Homes | Meas. Average Daily Flow | Daily Flow/ Home | |
| Door Creek | GRAVITY/SAND FILTER/BED | 1997 | 7 | 1,779 | 254 | 1,227 | -552 | 20 | 89 |
| Eagle Ravines | GRAVITY/BED | 1992 | 8 | 1,509 | 189 | 1,402 | -107 | 23 | 66 |
| Greenock Hills #3 | GRINDER/BED | 1990 | 8 | 1,679 | 210 | 1,341 | -338 | 22 | 76 |
| Hidden Ponds | GRINDER/BED | 1995 | 14 | 3,748 | 268 | 2,493 | -1,255 | 42 | 89 |
| Brooks River Landing | STEP/RSF/TRENCHES | 1994 | 15 | 1,051 | 70 | 2,311 | 1,260 | 39 | 27 |
| River Rock Landing | STEP/RSF/SURFACE WATER | 1987 | 19 | 1,935 | 102 | 2,928 | 993 | 49 | 39 |
| Highland Hills | STEP/BED | 1994 | 19 | 2,534 | 133 | 3,329 | 795 | 55 | 46 |
| Long Lake Pines | GRAVITY/SAND FILTER/BED | 1996 | 19 | 2,730 | 144 | 3,420 | 690 | 57 | 48 |
| Orchard Estates | GRINDER/BED | 1989 | 23 | 3,034 | 132 | 3,902 | 868 | 65 | 47 |
| Oaks At Beach Lake | STEP/SAND FILTER/BED | 1995 | 23 | 4,651 | 202 | 3,655 | -76 | 64 | 73 |
| Portage Bay Highlands | STEP/SAND FILTER/BED | 1995 | 27 | 3,206 | 119 | 4,731 | 1,525 | 70 | 41 |
| Sandy Creek | GRINDER/SAND FILTER/BED | 1996 | 34 | 5,082 | 149 | 5,542 | 460 | 92 | 55 |
| Runyan Lake | STEP/BED | 1987 | 183 | 23,250 | 127 | 32,588 | 9,338 | 543 | 43 |
| Tyrone Lake | STEP/BED | 1985 | 272 | 31,252 | 115 | 47,102 | 15,850 | 785 | 40 |

Total | 671 | 87,440 | 130 |

Note: Numbers in red above indicate projects where measured flows would have exceeded the estimated average daily flows using 60 GPD/capita and a population estimate based upon census data.

The number in black in the “Difference” column is the amount the estimated daily flow using a census estimate would have been larger than the actual average measured flows.
for the design flow from this typical residential neighborhood in southern Michigan, the peak daily flow exceeded that number by a factor of 1.6, or less, for these two full years of record!

**Conclusions**

The use of Census data from the smallest local jurisdiction in which a new residential community is to be located can be helpful information in predicting the expected population of that new community. Using that predicted population with the appropriate per capita flow is a rational way of determining projected wastewater flows from a watertight collection system for that community. Designer care should be taken to account for any expected lifestyle differences from the surrounding community being used as a comparison. The appropriate safety factor should be used to accommodate daily flow variations that will normally occur.

**References**


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GIS Mapping of Land Application Sites for Septage Management in Mahoning County, Ohio

AUTHORS
Wesley J. Vins, R.S., Dantan Hutton, R.S., John Bralich, B.A., and Matthew Stefanak, M.P.H.

Septage is defined as liquid or solid material removed from a septic tank, cesspool, portable toilet, type III marine sanitation device or a similar system that receives only domestic sewage (household, non-commercial, nonindustrial sewage) (Mahoning County, 2004). In 2004 the Mahoning County, Ohio, District Board of Health updated its Regulation of Servicing and Disposal of Septage to manage septic tank cleaning and the land application of septage. The rules were developed to address increasing public concern about the potential public health impact of allowing the application of hundreds of thousands of gallons of septage to farm fields in Mahoning County, while still allowing for a beneficial and economical method of disposing of septage.

In 2005 in Mahoning County, 2.8 million gallons of septage were pumped from household sewage treatment systems and a total of 927,226 gallons of septage was applied to eight Board-of-Health-approved land application sites; the remaining septage was either disposed of at a municipal wastewater treatment plant or land applied in another county. The significant increase in the amount of septage pumped since 1997 can be attributed to the Board of Health’s implementation of a successful pumping tracking and reminder program for the nearly 18,000 residences serviced by septic systems, pumpings which ultimately help to prevent the costly premature malfunction of septic systems but that also produce a larger volume of septage for disposal.

The Board of Health needed to provide tools to septage haulers to manage the additional septage in a manner that would be economical for the resident and hauler, and be beneficial to the community. Land application meets the needs of the community by providing for economical disposal in rural locations far from treatment plants, and it makes available a nutrient-rich soil building product to landowners.

The eight land application sites are operated and monitored in accordance with the 2004 Board of Health regulation, which requires site compliance with federal regulations governing land application of sewage sludge and septage, following guidance from the Ohio State University Extension (Ohio EPA, 1999, and OSU Extension, 1995). In an effort to assist the site operators in maintaining compliance, the Board of Health partnered with Youngstown State University (YSU) to identify the geographic boundaries of septage application in farm fields. Using a Geographic Information System (GIS) to map isolation distances is an excellent tool for regulators to determine available acreage and calculate loading limits for septage application, but this tool has limited value to the operator...
during actual field application. As a result, the Board of Health decided to convert mapped land application boundaries to Global Positioning System (GPS) coordinates and use these coordinates to locate application boundaries in the fields using hand-held GPS locators.

**Methods**

Aerial photographs outlining the boundaries of the land application areas in Mahoning County and a list of their corresponding GPS coordinates were provided to the Board of Health by the Center for Urban and Regional Studies at YSU using the Mahoning County GIS. These maps depict the proper isolation distances that land applicators must maintain from homes, streams, ponds, wells and property lines (Table 1).

Copies of the maps were then provided by the Board of Health to each site operator to assist them in locating the boundaries of land application on their sites. The sites marked during this project varied in size. The smallest site was 5.9 acres, which was permitted to apply 101,025 gallons per year, and the largest site was 34.8 acres, which can apply up to 834,232 gallons per year, depending on the expected crop rotation (Table 2).

The land application operators were first contacted and informed that the Board of Health would be visiting their sites to field locate the limits of septage land application in conjunction with the routine annual inspection. The site operators were requested to be present during the site visit to discuss the importance of the land application boundaries and the isolation distances. The site visit also allowed for an exchange of ideas for the best management practices to be implemented at each site.

The land application boundaries were field located by the Board of Health and the site operator using the Trimble GEO XT (Trimble Navigation Limited, Sunnyvale, CA) and the GPS coordinates provided by Youngstown State University’s Center for Urban and Regional Studies. These coordinates were obtained using ArcView GIS, version 3.3 (ESRI, Redlands, California) to overlay parcel boundaries, roads and water features with digital aerial photographs provided by the Mahoning County GIS. The Board of Health provided isolation distances from occupied dwellings, water, roads, and other sensitive features from its regulation to create maps of areas at each site where application of septage would be suitable.

To assist the Board of Health and the site operator in field locating the boundaries, points were mapped at several locations along the land application boundaries. Since the layers in each map were created using a State Plane projection to overlay the digital aerials, the coordinates of each point were converted to decimal degrees and given a letter for identification purposes. The decimal degree coordinates were then converted to degrees-minutes-seconds, a suitable format for the Trimble GEO XT, and entered into a spreadsheet. The acreage of each suitable location was also calculated using ArcView GIS to determine the allowable amount of septage applied at each site. Fieldwork was subsequently completed by the Board of Health and the site operator. Figure 1 contains an example of one mapped site. Each boundary point with corresponding GPS coordinates indicated on the map was marked in the field with survey tath and ribbon. Isolation distances due to sensitive areas

<table>
<thead>
<tr>
<th>Table 1</th>
<th>Isolation Distances for Land Application of Septage—Mahoning County General Health District.</th>
</tr>
</thead>
<tbody>
<tr>
<td>• Field drainage swales, wet weather and intermittent streams – 33 feet</td>
<td></td>
</tr>
<tr>
<td>• Lakes, ponds, rivers, creeks – 500 feet</td>
<td></td>
</tr>
<tr>
<td>• Occupied dwellings – 500 feet</td>
<td></td>
</tr>
<tr>
<td>• Wells and cisterns – 500 feet</td>
<td></td>
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<tr>
<td>• Property lines, road – 50 feet</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Table 2</th>
<th>Eight Sites Approved for Land Application of Septage in Mahoning County</th>
</tr>
</thead>
<tbody>
<tr>
<td>Site</td>
<td>Acres</td>
</tr>
<tr>
<td>A</td>
<td>16.9</td>
</tr>
<tr>
<td>B</td>
<td>19.6</td>
</tr>
<tr>
<td>C</td>
<td>32.8</td>
</tr>
<tr>
<td>D</td>
<td>34.8</td>
</tr>
<tr>
<td>E</td>
<td>6.8</td>
</tr>
<tr>
<td>F</td>
<td>5.9</td>
</tr>
<tr>
<td>G</td>
<td>7.2</td>
</tr>
<tr>
<td>H</td>
<td>21.6</td>
</tr>
</tbody>
</table>
such as homes, streams, ponds, and lakes received additional field staking. The marked boundaries clearly defined a visual limit of application to the operator, which assists them during actual septage application. Every field-located point was stored on the hand-held GPS locator for future reference. The stored coordinates make it possible to navigate to marked boundary points very quickly and reestablish a visual limit of application.

During the visits, the regulators and operators can visually identify the application limits of their site and discuss best practices for land application within these limits. Disposing of septage using land application is an economical method, which has many benefits for generating productive soils that are capable of producing various crops. The crops grown on the Mahoning County sites include hay, corn, barley, wheat, and soy beans.

**Results**

The GIS data generated by YSU and field located by the Board of Health assisted five operators in visually locating the limits of septage application at eight sites. The marked boundary limits help ensure the proper application of the nearly 900,000 gallons of septage within the boundaries established by the isolation distances. The defined limits of application help regulators address occasional community concerns about the safety of this practice and provide an opportunity to discuss management practices with the operator. The marked limits of application have also provided the operator information necessary to maintain compliance while land applying septage.

**Conclusions**

We have used this GIS and GPS technology to assist a local company and group of concerned citizens living near a proposed site come to an agreement that the site did not have adequate acreage for economical land application, thereby saving the company from making a significant financial investment in order to acquire the site. GIS and GPS technology can help regulators to calculate accurate septage loading limits for land application sites and provide site operators with visual boundaries to safely and effectively apply septage without causing unnecessary environmental hazards that may result in a threat to public health. Site visits by regulators provide an opportunity to discuss many land application issues with operators, including the importance of isolation distances and their corresponding limits of application.

**References**


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