Evaluation of Septic Tank and Subsurface Flow Wetland for Jamaican Public School Sewage Treatment

By

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A REPORT

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This report "Evaluation of Septic Tank and Subsurface Wetland for Jamaican Public School Sewage Treatment" is hereby approved in partial fulfillment of the requirements for the degree of MASTER OF SCIENCE IN ENVIRONMENTAL ENGINEERING.

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Abstract

Locally designed wastewater treatment systems at two rural Jamaican public schools located in Pisgah, St. Elizabeth and Retrieve, St. James, were evaluated over a seventeen week period to evaluate their effectiveness. Primary treatment was achieved with two plastic tanks in series. This was followed by a horizontal subsurface flow constructed wetland planted with local wild cane (*Gynerium sagittatum*) achieving secondary treatment with nutrient removal. (In Jamaica, this is referred to as tertiary treatment.) The toilet system in Pisgah had been operating for one and a half years supplied with water from a rainwater harvesting scheme. Per capita water use averaged 1.3 L/p-d and total water use averaged 264 L/d (70 USgal/d). This resulted in such low hydraulic loading that the septic tanks had a hydraulic residence time (HRT) of 29 days. The wetland never produced effluent so that it functioned as an evapotranspiration (ET) bed with an average ET rate of 0.27 USgal/ft^2/d (11 mm/d). Wetland BOD mass loading averaged 0.79 kg/ha-d (0.71 lb/ac-d) and TSS entry zone mass loading averaged 0.0013 lb/ft^2-d (6.4 g/m^2-d). The toilet system in Retrieve had been operating for two years with a municipal water supply. Per capita water use averaged 48.3 L/p-d and total water use averaged 3,240 L/d (857 USgal/d). The average septic tank HRT was 1.2 days. Combined sewage inflow and precipitation to the wetland resulted in an average HRT of 2.2 days equivalent to a hydraulic load of 2.6 USgal/ft^2-d (105 L/m^2-d). Wetland BOD mass loading averaged 15 kg/ha-d (13 lb/ac-d) and TSS entry zone mass loading averaged 0.0038 lb/ft^2-d (19 g/m^2-d).

Five sets of water quality grab samples were collected from three points at each site. Samples from the sanitation system in Pisgah indicated an average reduction of raw sewage BOD by 78%, TSS by 85%, total nitrogen by 95%, ammonia by 99%, total phosphorus by 97%, total coliform and fecal coliform by 4 log (99.99%). Samples from the sanitation system in Retrieve indicated an average reduction of the raw sewage BOD by 50%, total nitrogen by 68%, ammonia by 97%, total phosphorus by 64%, total coliform by more than 3 log and fecal coliform by more than 4 log. An increase in TSS for the system at Retrieve may have been caused by the effluent sampling method. Average nitrate levels were below 1 mg/l throughout both sanitation systems.

Preface

This report was prepared from research conducted at the National Water Commission laboratory in Bogue, St. James, Pisgah All Age School in St. Elizabeth and Retrieve All Age School in St. James, Jamaica during the latter half of 2004. Permission was granted from the National Water Commission Western Division Quality Assurance Manager, Richard Meggo; Eastern Division Quality Assurance Manager, Don Streete; the Principal of Pisgah All Age School, Ms. Simms; and the Principal of Retrieve All Age School, Mr. Scott.

Research and field work was coordinated with the following people: my advisor, Dr. Jim Mihelcic of Michigan Technological University; my supervisor, Richard Meggo of the National Water Commission; engineer Jason Excell and logistics coordinator Neville Williams both of the Ridge To Reef Watershed Project.

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Chapter 1 - Introduction

Onsite Sanitation Technology

For many years the common onsite sanitation technology in Jamaica for handling residential and commercial black and gray water has been the "absorption pit" (see Figure 1-1). This is a large hole in the ground that is capped with a concrete slab into which all kitchen gray water and toilet wastewater empties. Depending on the specific hydraulics of each site, the absorption pit functions as a sealed vault or subsurface wastewater infiltration system. The benefit is that there is no public exposure to open sewage, but the geology in Jamaica and geometry of the pit often result in inadequate sewage treatment and groundwater contamination.

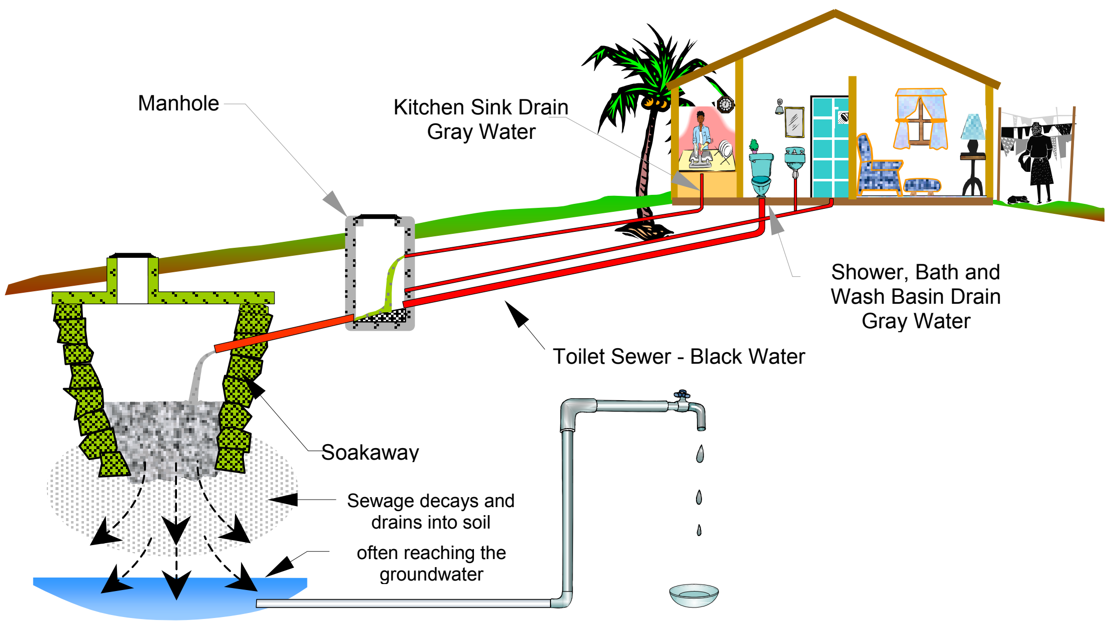

Figure 1-1 Absorption pit soakaway system. (Illustration from Gray, 2001, courtesy of Associates in Rural Development.)

The predominant geological formations in Jamaica consist of White Limestone, Yellow Limestone and Volcanics (Smikle, 2000). Locations with porous soil structures allow raw sewage to empty into the groundwater table. The geometry of a wastewater infiltration system affects how well the water is dissipated and treated by the soils. Infiltration trenches that are long, narrow and shallow are preferred over wide beds and deep pits. A biological mat of microbes forms as wastewater flows into the soil. This mat and cover vegetation capture and consume the organics and nutrients making it possible for high reduction of biochemical oxygen demand (BOD), total suspended solids (TSS), phosphorus, viruses and coliform bacteria. The nitrate form of nitrogen is not readily removed by soil so nitrate levels may be elevated under any system that does not adequately remove it prior to infiltration (USEPA, 2000). Although some biological activity is expected in a deep pit infiltration system such as the Jamaican absorption pit, the predominant geology and geometry are in no way favorable to adequate treatment.

An absorption pit can be dug by hand or may require a jackhammer to break through large rocks or solid rock. If the walls of the excavation are not stable, rocks are stacked from the bottom up to create a wall of uncemented rocks lining the entire hole. Depths range from six to twenty feet and widths vary from five to eighteen feet with the top normally being the widest and tapering toward the bottom. Absorption pits are expected to fill up after years of use at which time another pit is dug. Some cover slabs are designed with a manhole so the septage can be pumped out when required. At present there are only two National Water Commission (NWC) waste treatment plants that regularly receive and treat septage.

A natural variant of the absorption pit is a sink hole (locally called a "say ball" or "say bowl") in the limestone that connects to an underground void or aquifer. Household sewage pipes are connected to such a hole which is then capped with concrete to create a permanent waste water disposal system. This eliminates the expense and labor of digging a pit and greatly increases the life of the sewage disposal system since it is unlikely that the sink hole will ever fill up.

In Jamaica, all onsite sanitation system design and construction plans are supposed to be approved by the local Parish Council. Developments greater than ten housing units must be approved by the National Environment and Planning Agency (NEPA) and Ministry of Health. The Water Resources Authority (WRA) contributes to the decision making process by providing recommendations which promote the sustainability of Jamaica's water resources. "Given the capability of absorption pits to contribute to water contamination the WRA is cautious in approving this system as a method for sewage disposal. However where groundwater resources are at great depth [200m], of insignificant quantity, regional faulting is not extensive and surface water and surface water channels are not nearby then absorption pits may be employed. It should be noted that the WRA recommends the installation of a Septic Tank prior to final discharge in the absorption pit where primary treatment level systems are suitable"(Pennant, 2005). The septic tank with absorption pit may be approved under certain conditions, but the absorption pit alone is never an appropriate solution for domestic sewage.

Groundwater Contamination

A typical example of coliform contaminated groundwater in a rural area was observed during a visit to an entombed spring in upper Trelawny Parish. The Water Quality Officer reported that water from this spring had a history of fecal coliform contamination greater than 2,400 MPN/100ml (Morris, 2003). There is no protected zone uphill of the springbox, but rather a butcher shop and several houses within two hundred feet. A nearby absorption pit and pit latrine are obvious sources of fecal coliform. Pit latrines may contaminate groundwater to a lesser degree than absorption pits because the reduced fluid volume entering the latrine results in a weaker driving force transporting the waste.

The national effluent standards for municipal wastewater discharged in Jamaica have been changing in recent years. The increased stringency in nutrient concentration and coliform limits reveals growing awareness and concern for how these affect public health. Table 1-1 compares the discharge standards for municipal wastewater treatment plants discharging a volume of $20 m^3/d$ (5,300 USgal/d) or more that were built prior to 1997 and the proposed standards for newer plants. Clause 10.4 of the December 2001 revision of the National Sewage Effluent Regulations states that the grandfather period should end on January 1, 2005 so that all large plants must meet the new standards. Smaller municipal systems are to report coliform and flowrate once a year. Jamaica has a policy similar to most of the United States where small onsite sanitation systems are design based and not performance based so they are not tested to meet specific effluent standards (USEPA, 2002).

Table 1-1 **Jamaican National Sewage Effluent Standards for Municipal Systems Discharging 20 m^3/d (5,300 USgal/d) or More.**

PARAMETER	1996	1997 - Present
BOD_5	20 mg/l	20 mg/l
TSS	30 mg/l	20 mg/l
Nitrates (as Nitrogen)	30 mg/l	n/a
Total Nitrogen	n/a	10 mg/l
Phosphates (as Phosphorus)	10 mg/l	4 mg/l
COD	100 mg/l	100 mg/l
pH	6 - 9	6- 9
Fecal Coliform	1,000 MPN/100ml	200 MPN/100ml
Residual Chlorine	1.5 mg/l	1.5 mg/l

The following two citations provide some details on urban area groundwater contamination observed in Jamaica.

> "In the Kingston Metropolitan Area, KMA, the alluvium aquifer is contaminated by nitrate from sewage soakaway systems/pit latrines. Over 40% of the aquifer is not suitable for use as a domestic supply. The NWC has abandoned over 8 MCM/yr [eight million cubic meters per year] of well production from the alluvium aquifer. Recent data has indicated that the NWC limestone wells in the Kingston

Metropolitan Area are showing increased nitrate concentration, moving from 4 mg/l in 1968 to 23 mg/l in 1993. Housing development using soakaway systems continue, unabated, hydraulically upgradient of the wells" (Fernandez, 1993).

"A preliminary literature review gave some indication of the types of impacts on groundwater quality and informed the process of selecting pollution indicators. With the primary impacts being from sewage disposal through use of absorption pits and saline intrusion exacerbated by over pumping, the pollution indicators selected were total and faecal coliform bacteria, nitrate, chloride. Nitrate is the principal form of combined nitrogen found in natural waters. Most surface waters contain some nitrates, however concentrations greater than 5 mg/l may reflect unsanitary conditions, since one major source of nitrates is human and animal excrement. The consumption of waters with high nitrate concentrations decreases the oxygen carrying capacity of the blood. This is particularly important in the health of young infants, who may develop methemoglobinemia. An association between the nitrate contamination in wells and their proximity to high density population areas is clear. All the wells which fall within Grade D – advanced deterioration (nitrates >80mg/l) are located just south of or on the southern boundary of districts with population densities between Class 6 (10,001-12,000 persons/km^2) and Class 9 (>24,000 persons/km2). The fact that many Grade D wells are actually located in low populations suggests that nitrates are transferred throughout the aquifer via groundwater movement with the natural hydraulic gradient, in this case generally southern direction. All Grade A (NO_3 2-8mg/l) and B (NO_3 >8-45mg/l) wells are located in and south of districts with lower populations densities" (Underground Water Authority, 1996).

Engineering Objective

According to the World Health Organization's 2000 report, Latin America and the Caribbean have 51 million people in urban areas and 66 million people in rural areas that lack access to acceptable excreta disposal facilities. Worldwide there are 403 million people in urban areas and 2 billion people in rural areas that lack improved sanitation (WHO, 2000). There is a need for appropriate onsite sanitation technology in Jamaica that is effective in treating contaminants, cost effective and easy to build and operate in rural and urban settings. The Ridge To Reef Watershed Project (R2RW) developed an onsite sanitation system using septic tank and subsurface flow constructed wetland technology to reduce the environmental impact of wastewater in the Great River and Rio Grande watersheds of Jamaica. Although this technology was accepted internationally, it needed to be proven in the local context before it could become widely accepted in Jamaica. Accordingly, an evaluation was needed to document treatment effectiveness and the impact of design parameters in local conditions so more systems could be built for optimum performance.

In response to this need, a proposal was presented to and approved by the R2RW Water and Sanitation Committee to conduct an engineering evaluation of two of these onsite sanitation systems that had been operating for more than a year and a half. The evaluation included physical measurements, observation of operating conditions and water quality testing. This report contains a discussion of the data and suggestions for design improvements based upon field observations and current engineering literature.

Chapter 2 – Unit Operation Design Parameters

Septic Tanks

Septic tanks (see Figure 2-1) separate solids from liquids by gravity settling so the effluent has reduced BOD and TSS. Anaerobic decomposition will reduce the volume of accumulated solids on the bottom of the tank by 40 – 50% producing methane, carbon dioxide, water and reduced sulfur gases (Seabloom et al., 1982; USEPA, 2000). Inorganic solids (i. e. sand) and undigested organic solids form sludge in the bottom of the tank that should be removed before accumulation reaches a volume that hinders performance. The scum layer formed of soaps, oils, greases and light debris should also be removed before it becomes too thick and passes around the sanitary tee and out of the septic tank.

Performance of septic tanks depends on influent characteristics and tank design. Septic tank removal efficiencies have been reported as follows: BOD 46 – 68%, TSS 30 – 81%, phosphate 20 – 65%, fecal coliform 25 – 66% (Seabloom et al., 1982; Rahman et al., 1999).

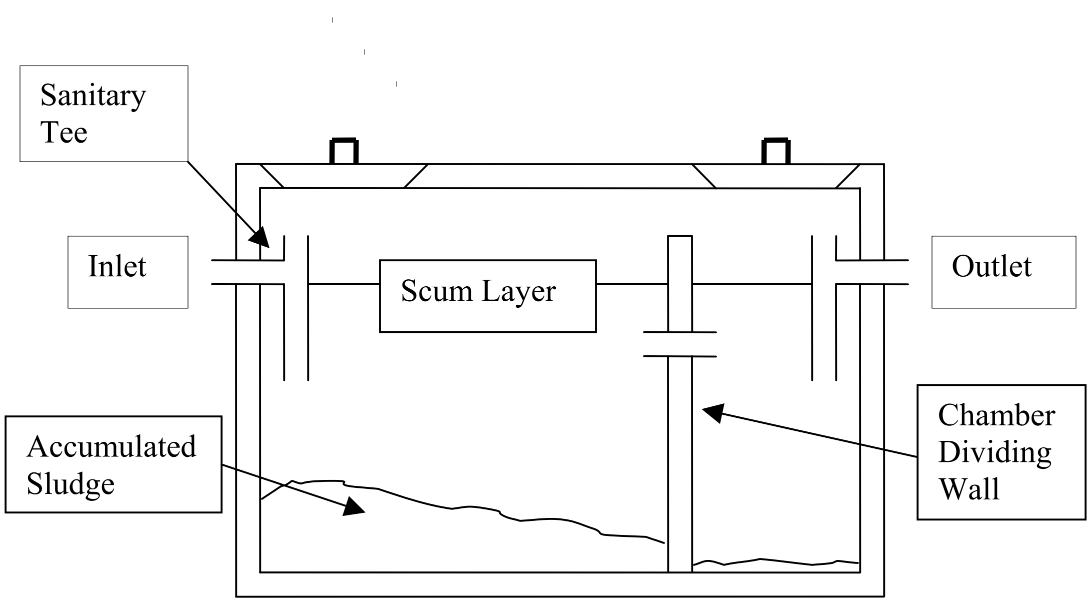

Figure 2-1 Typical concrete two-chamber septic tank.

Critical design parameters for septic tanks include proper piping, tank volume and tank dimension. A sanitary tee pipe fitting on the inlet directs the influent downward to reduce short circuiting. A sanitary tee on the outlet prevents floating scum from exiting and clogging the wetland or tile field receiving the effluent. For a two-chamber tank, the chamber dividing wall allows liquid free of scum and sludge to pass from the first chamber to the second chamber, and it has ventilation above the liquid level to allow pressure equalization between the chambers. Sufficient tank volume is necessary so that after the maximum expected volume of sludge and scum has accumulated, the tank has a minimum of 24-hour fluid retention time for particulate settling (USEPA, 2000). Rules of thumb for septic tank volume range from two to five times the daily average flow (USEPA, 1980; Crites et al., 1998). A peaking factor should be used to account for the irregular pattern of flow into the tank during peak water use periods. Two-chamber septic tanks are believed to have an advantage over single-chamber tanks because of the reduced turbulence in the second chamber which provides more quiescent settling conditions. Although the second chamber has a more dampened rate of inflow, proper tank sizing and dimension are still necessary to achieve improved effluent quality.

The effect of tank dimension was studied by comparing a single-chamber and a two-chamber septic tank of equal volume. BOD and TSS removal were reported to be better in the single-chamber tank. The reason was attributed to the surface area in the chambers and resultant overflow rate which directly impacts discrete particle settling. Because the single-chamber tank was not divided in two, it had a larger surface area with slower overflow rate than the two-chamber tank (Seabloom et al., 1982). The larger surface area also reduces the head differential caused by influent surge and dampens the effect on exit velocity which if too high could draw suspended particles out of the tank (USEPA, 1980). However, other studies showed better solids removal efficiencies in two-chamber tanks because settled solids in single-chamber tanks were resuspended by bubbles rising from the anaerobic digestion of the accumulated solids (Laak, 1980; Rock and Boyer 1995; Rich, 2006 as referenced in WERF, 2007). A single-chamber tank with effluent filter that is periodically cleaned performs better than a multiple-chamber tank without a filter

(Crites et al., 1998). Design must balance the need for large surface area to create slow overflow rates that promote ideal particle settling, sufficient depth to store settled solids and chambers to prevent resuspension of settle solids or an effluent filter.

Septic tanks in Jamaica are usually constructed of a concrete slab floor with concrete block walls, and the inside is rendered watertight with neat cement. The R2RW septic systems were designed according to the two-chamber septic tank principle, but construction costs were reduced by using two separate cylindrical plastic tanks (see Figure 2.2). These tanks were very similar to the water storage tanks seen on rooftops in Jamaica, but the wall thickness was doubled and piping connections were customized. The tank sizing was based on the desire to have a 3 day HRT with an expected water use of 10 USgal/person per day (USgal/p-d).

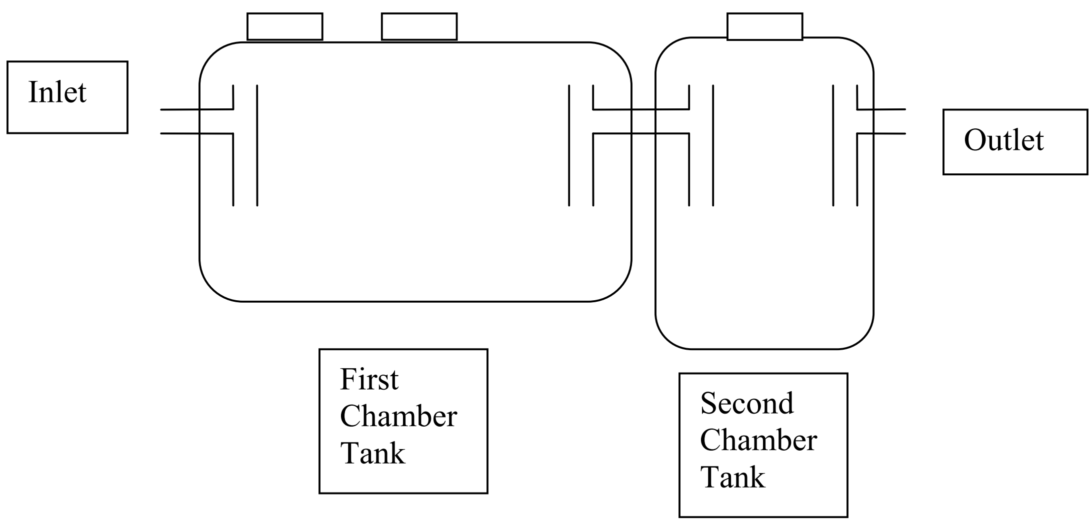

Figure 2-2 Ridge To Reef Watershed Project two-chamber septic tank made from plastic tanks.

Subsurface Flow Constructed Wetlands

A constructed wetland is a simulated natural environment for the filtration and biological treatment of wastewater. Horizontal subsurface flow (SSF) wetlands are shallow beds filled with rocks and are usually planted with aquatic vegetation (see Figure 2-3). They are designed to keep the liquid level 3 to 4 inches below the surface of the rock media to prevent public exposure to the wastewater and mosquito breeding.

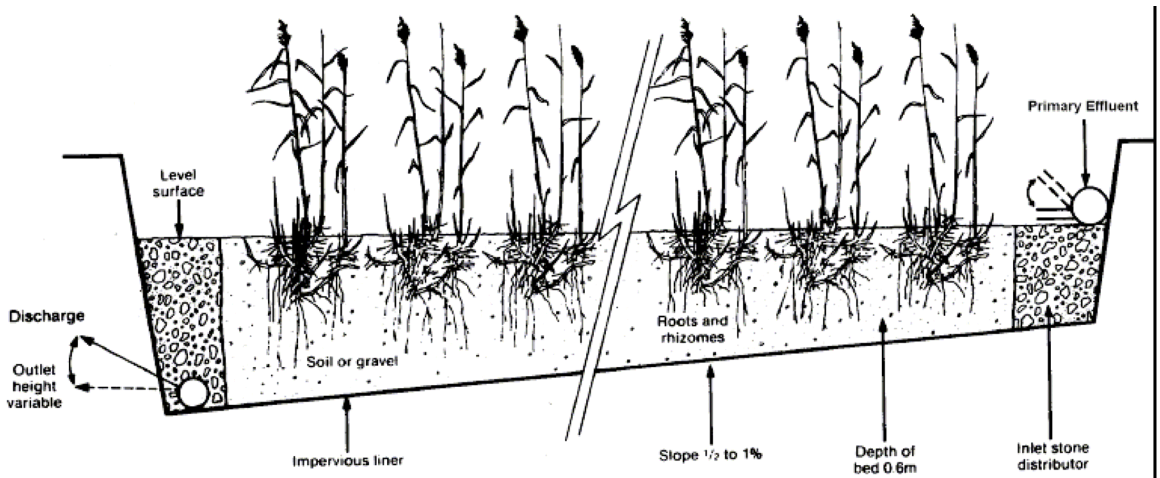

Figure 2-3 Side view of a horizontal subsurface flow wetland showing vegetation. (Illustration by Cooper, 1993. Republished with permission.)

SSF wetlands are a widely accepted technology for reducing wastewater BOD, TSS and coliform (Crites et al., 1998; USEPA, 2000; Reed et al., 2001). Several mathematical models are available for calculating the wetland surface area or volume necessary to achieve the required BOD and nitrogen removal. TSS treatment follows closely with BOD removal so that designing for BOD parameters will provide sufficient treatment to meet similar TSS treatment standards (Reed et al., 2001). According to one authority, a wetland designed with 6-10 day HRT, BOD mass load below 100 lb/ac-d (89 kg/ha-d) and influent nitrogen below 25 mg/l should meet effluent standards of 20 mg/l BOD, 20 mg/l TSS and 10 mg/l N (Crites et al., 1998). Removal efficiencies based on inflow and outflow concentrations through horizontal SSF wetlands in Europe and North America have been reported to range between 68.5-92.7% for BOD, 63.0-89.8% for TSS, 26.7-65.0% for phosphorus and 60.1-64.8% for nitrogen (Vymazal, 2002).

There is significant complexity in the biological, physical and chemical mechanisms responsible for pollutant removal in SSF wetlands so that the proposed models do not fit all the data collected. Accordingly, until further understanding is gained the USEPA recommends limiting mass loading rates to those proven in the field. Subsurface wetlands in the United States with maximum monthly TSS loads below 200kg/ha-d (178 lb/ac-d) and maximum monthly BOD loads below 60 kg/ha-d (54 lb/ac-d) have consistently provided effluent below 30 mg/l TSS and 30 mg/l BOD. Nitrogen removal was not found to consistently correlate with loading limits (USEPA, 2000).

Proper hydraulic design takes into account the maximum expected volumetric flowrate, maximum expected precipitation, bed geometry, media porosity and increase in flow resistance caused by accumulation of solids. Elevations must be known to account for head change in the wetland and the head needed at the outlet so effluent will exit. If the site topography promotes rainwater runoff or other surface water to travel toward the wetland, an earth berm around the wetland may be necessary to prevent surface water from entering. The design should prevent plant roots from clogging the inlet distribution pipes. Multiple beds in parallel are recommended for operational flexibility and adjustable outlets are recommended for controlling the water level. Length-to-width (aspect) ratio may be adapted to accommodate available land space but should not be too long and narrow which promotes surface flooding at the inlet. Even distribution of flow across the width of the wetland is necessary to reduce short circuiting.

The media should be hard rock or stone that is washed to prevent fines from building up and blocking flow. Typical gravel size in the United States is 0.75-1.0 inch (19-25 mm) diameter. Recent designs incorporate larger diameter rocks of 2 to 6 inch (51-152mm) diameter at the entry and exit zones of the wetland bed to facilitate fluid dispersion and prevent plant root encroachment (USEPA, 2000). However, the transportation cost for bringing larger rock from a distant quarry was found to be cost prohibitive during a recent small wetland construction project in Western Jamaica. Wetlands with gravel media receiving primary effluent at low solids loading should not clog or require solids removal for many years (Crites et al., 1998; Reed et al., 2001). Gabions (i. e. wire cages packed with rocks) may be used at the inlet to simplify removal and cleaning if excessive accumulation of solids is expected over time. Most TSS accumulates in the first 20% of a SSF wetland bed. One recommendation has been made to design SSF wetlands based on a limit of 0.008 lb/ft^2-d (39 g/m^2-d) for TSS mass loading at the entry zone to prevent media clogging (Crites et al., 1998). The TSS entry zone mass loading is determined as follows: TSS Load = (TSS mass/day) / (Entry Zone Cross Sectional Area).

Large wetlands are often designed with a sloped floor to provide complete drainage and provide the necessary head to overcome an estimated hydraulic conductivity. For a flat floor wetland, the height of water will be greater at the inlet (see Figure 2-4), and the media depth should be increased at the inlet end to account for the higher water level.

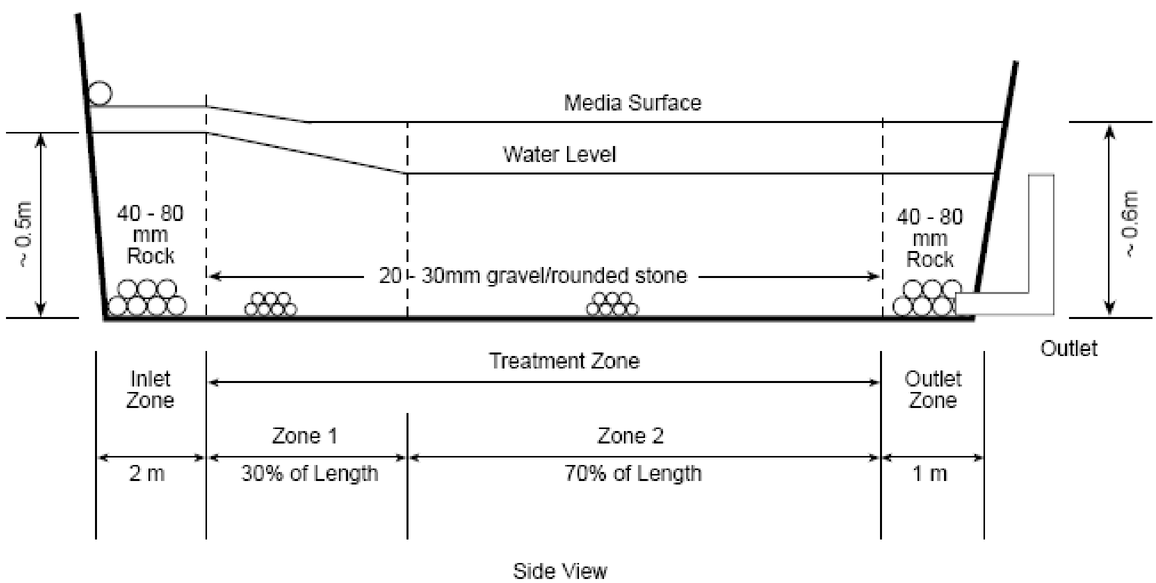

Figure 2-4 Side view of a horizontal subsurface flow wetland showing media distribution and water level. (Illustration by USEPA, 2000. Public domain.)

Small wetlands for single-family sanitation systems may use a variety of plants that improve water treatment and add color to the water garden. Plants such as the soft rushes (*Juncus balticus*, *Juncus effuses*), umbrella palms (*Cyperus alternifolius, Cyperus papyrus*) and woolgrass (*Scirpus cyperinus*) have deep roots and do not grow too tall. These are beneficial characteristics. Graceful cattail (*Typha laxmanii*) is a dwarf variety that does not grow too tall but has shallow roots. Iris (*Iris versicolor*) and thalia (*Thalia dealbata*) provide attractive blooms and foliage variety, but may not significantly enhance water treatment (Schellenberg, 2001).

Larger SSF wetland beds may use native aquatic plants such as rushes (*Juncus*), cattails (*Typha*), reeds (*Phragmites*) and bulrushes (*Scirpus*) that are tolerant to flooding and wastewater. One study indicated that broadleaf cattails may outperform some rushes and bulrushes in improving effluent quality, and having more than one type of plant may improve system performance (Coleman, et al., 1999). No routine cutting or harvesting is required to operate a wetland, but regular harvesting will increase nutrient removal from the effluent (Koottatep et al., 1997; USEPA, 2000).

The R2RW wetlands evaluated were sized for an expected hydraulic loading of 1 USgal/ft^2. No slope was built into the floors. The beds were filled with ½-inch stone to a depth of approximately 20 inches and planted with wild cane (*Gynerium sagittatum*). Parallel beds were built at each site to provide operational flexibility. Wetland effluent pipes were fit with PVC elbows that could be turned so the bed water level could be controlled.

Chapter 3 – Site Descriptions

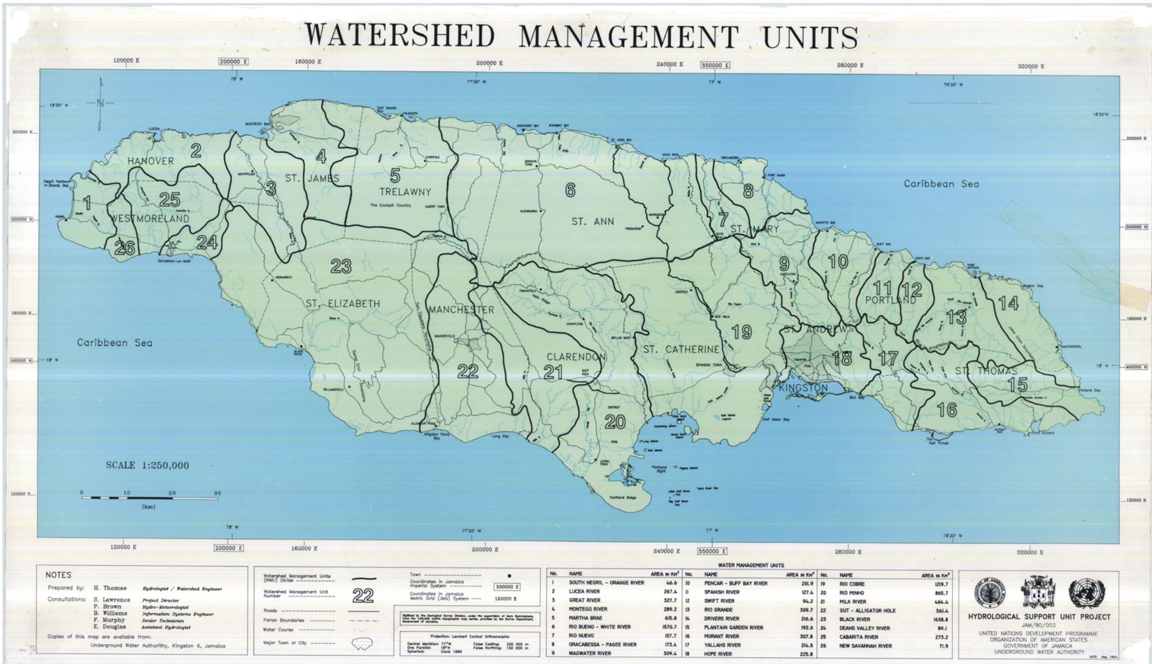

Figure 3-1 Jamaica's watersheds with the Great River watershed in the northwest labeled as "3." (Map courtesy of Water Resources Authority, Kingston)

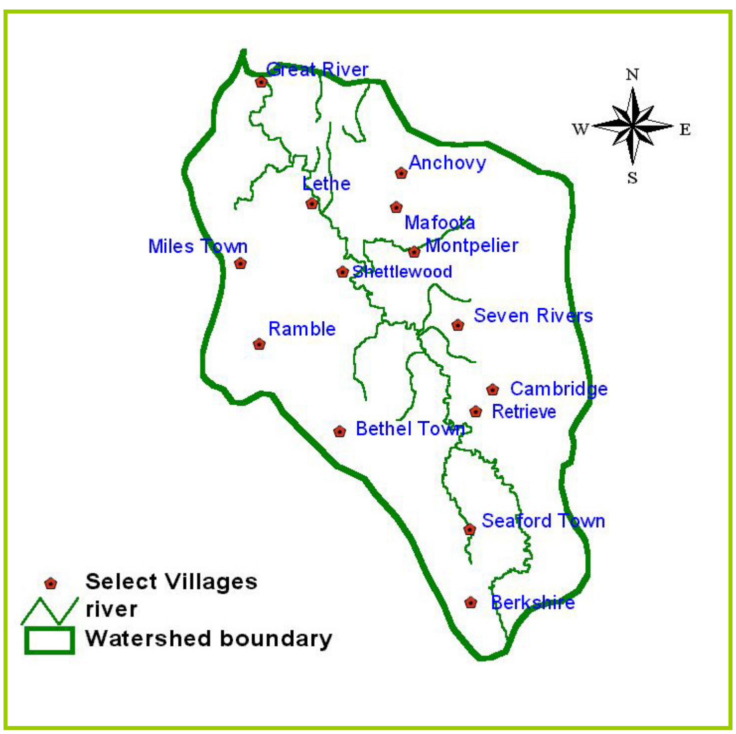

Figure 3-2 Jamaica's Great River watershed. (Map courtesy of Ridge to Reef Watershed Project, National Environment and Planning Agency, Kingston.)

Five septic tank/wetland systems had been built by R2RW in the Great River watershed (see Figure 3-1) at the time of the evaluation, but only two were known to be consistently operating. These two systems were at the Pigsah All Age School in St. Elizabeth Parish and the Retrieve All Age School in St. James Parish, both of which are in the northwestern region of the island. Pisgah is located southeast of Berkshire, and Retrieve is southwest of Cambridge (see Figure 3-2). The three sanitation systems that were not evaluated were at an infrequently used community center and two single-family dwellings.

Pisgah Sanitation System Site Description

The Pisgah All Age School sanitation system (see Figure 3-3) commenced operation in January 2003 and serves one shift of approximately 205 students five days per week. The septic tanks and wetland treat only black water from the toilets and no gray water. The hand washing station located between the toilets and the school drains to the ground rather than the septic system. There were originally two pit latrine buildings each with four seats. Three pit seats in each building were converted into flush toilets supplied with water from the school's rainwater harvesting storage tanks. One pit latrine was left in each toilet building for use on those days when piped water is not available.

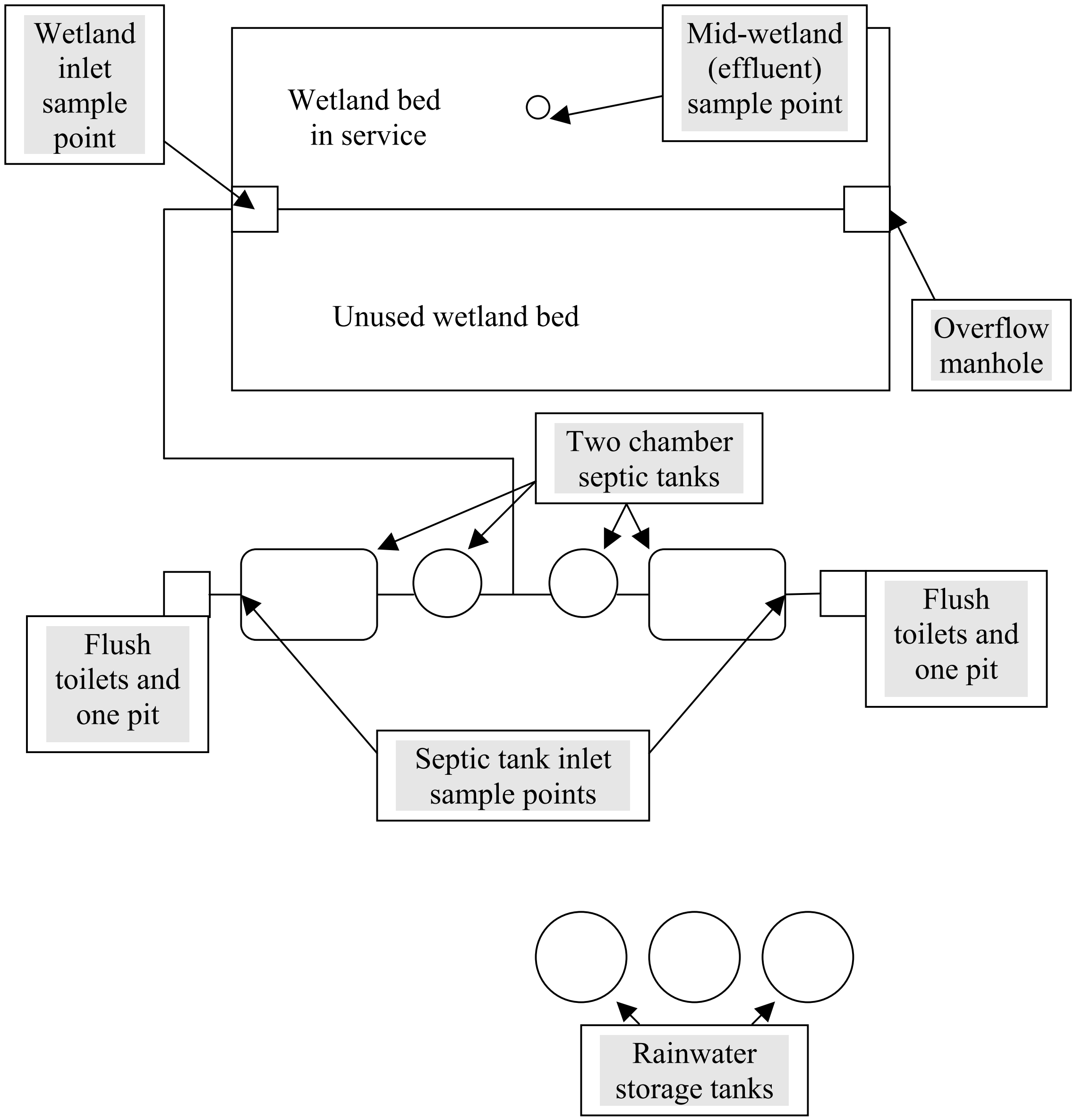

Figure 3-3 Schematic of Pisgah sanitation system showing sampling locations.

The toilets in the boys' block flush through 4-inch PVC pipe into a horizontal 880-US gallon (3,326 liter) polyethylene tank 7 feet 3 inches (221cm) long and 4 feet 6 1/2 inches (138cm) in diameter. The 880-gallon tank empties into a vertical 400-gallon (1,512 liter) polyethylene tank 5 feet 6 inches (168cm) tall and 3 foot 8 inches (112cm) in diameter. The wall thicknesses of the 880-gallon and 400-gallon tanks are 7/16 inch (11mm) and

3/8 inch (9.5mm) respectively. This is twice the thickness for similar tanks used to store water on house rooftops. The girls' toilet block empties into an identical arrangement of tanks that are on the same plane as the boys' toilet tanks. All tanks are buried with access through manholes and all connections are 4-inch PVC with sanitary tees at tank inlets and outlets. In an effort to reduce cost yet maintain effective primary treatment (i. e. solids settling), there was no divider wall built into the tanks, but the effect of a two chamber septic tank was achieved by the use of two tanks. The volume of water contained in each two-chamber septic tank was estimated to be approximately 1,024 US gallons (3,870 liters) equivalent to 80% of total tank volume. The effluent from the septic tanks travels down approximately 6 vertical feet and 40 horizontal feet to the inlet distribution manhole of the SSF wetlands.

During design and construction, the design engineer expected the BOD mass loading to be high enough to demand aeration, so electric powered snorkel pump aerators were installed in both of the 880-gallon tanks. However, an anaerobic system such as a septic tank should adequately treat normal household or school wastewater, so the aerators were disconnected several weeks prior to the evaluation. This allowed the evaluation to be conducted on a non-electric system which might be more appropriate for widespread use in Jamaica. It was interesting to note that the first set of trial-run samples collected from the 880-gallon tanks contained a floating mass that looked like tiny, dry plant seeds. Inspection with a microscope revealed this to be dead crustaceans and rotifers. This must have been a thriving community before the oxygen supply was cut off.

Figure 3-4 Pisgah wetland beds before planting. The black plastic liner running down the middle separates the two beds. (Photograph by author.)

The wetland has two parallel plastic lined rock media beds (see Figure 3-4) each with inside dimensions of 62.5ft (19.1m) length x 15.5ft (4.7m) width x 1.6ft (0.5m) depth of media. Media is irregular shaped washed stone ordered as “1/2 inch river shingle” from a quarry on the south side of the island. The bulk of the stone ranges in size from ¼ to 1 inch (6 -25 mm) diameter. The media without plant roots had a measured porosity of 37.7%.

The bed in use was planted around October 2003 making the wild cane ten months old at the beginning of the evaluation. Ten months is long enough for individual plants to become mature, but new growth will spread to cover the available space so that the foliage may be considered young and in a rapid growth stage. The alternate parallel bed is unplanted and has never been in service. The 969 ft^2 (90 m^2) size of each bed was based upon a higher hydraulic loading estimate than what is actually received. The frugality of water use inspired by sole dependence on rainwater resulted in extremely low water use per capita, so the oversized wetland has never produced effluent and thus functions as an evapotranspiration bed.

Figure 3-5 Pisgah wetland at the beginning of the evaluation with ten month old wild cane. The design engineer is standing at the mid-wetland sample point used for collecting the "effluent" sample. (Photograph by author.)

The plants at the inlet end of the wetland (left side of Figure 3-5) were significantly higher and greener than the cane at the outlet end. The cause for the abrupt difference in plant height was not determined. The entire bed had good sun exposure. The tall cane grew to approximately 12 feet (3.7 m) with some shoots up to 15 feet (4.6 m) high. After Hurricane Ivan struck in September 2004, this cane was bowed over from the wind so it was cut to a height of 10 - 30 inches (25 – 76 cm). One sucker shoot, measured three times over a two-week period, grew from a height of 56.5 to 72.5 to 102.7 inches. This average growth rate of 3.3 inches (8 cm) per day is significantly faster than the reported seedling growth rate of 10 cm per month or 0.33 cm per day (Kalliola et al., 1991). The sucker grows faster because it already has established roots.

Retrieve Sanitation System Site Description

The Retrieve All Age School sanitation system (see Figure 3-6) began operation in September 2002 and serves one shift of approximately 69 students five days per week. The septic tank and wetland treat only black water from the toilets and no gray water. Water from the hand wash stations drains to the ground and not to the septic tank or wetland. Three pit latrine buildings (see Figure 3-7) were modified to accommodate flush toilets. A building with wash basins and running water was usually locked so students used an outdoor spigot next to this building. During the evaluation period two latrine buildings with four toilets each were in use as well as a urinal that had flush water running constantly. Water usually comes from the NWC Dantrout groundwater source. A rainwater harvesting scheme was built with the sanitation system provides water on occasions.

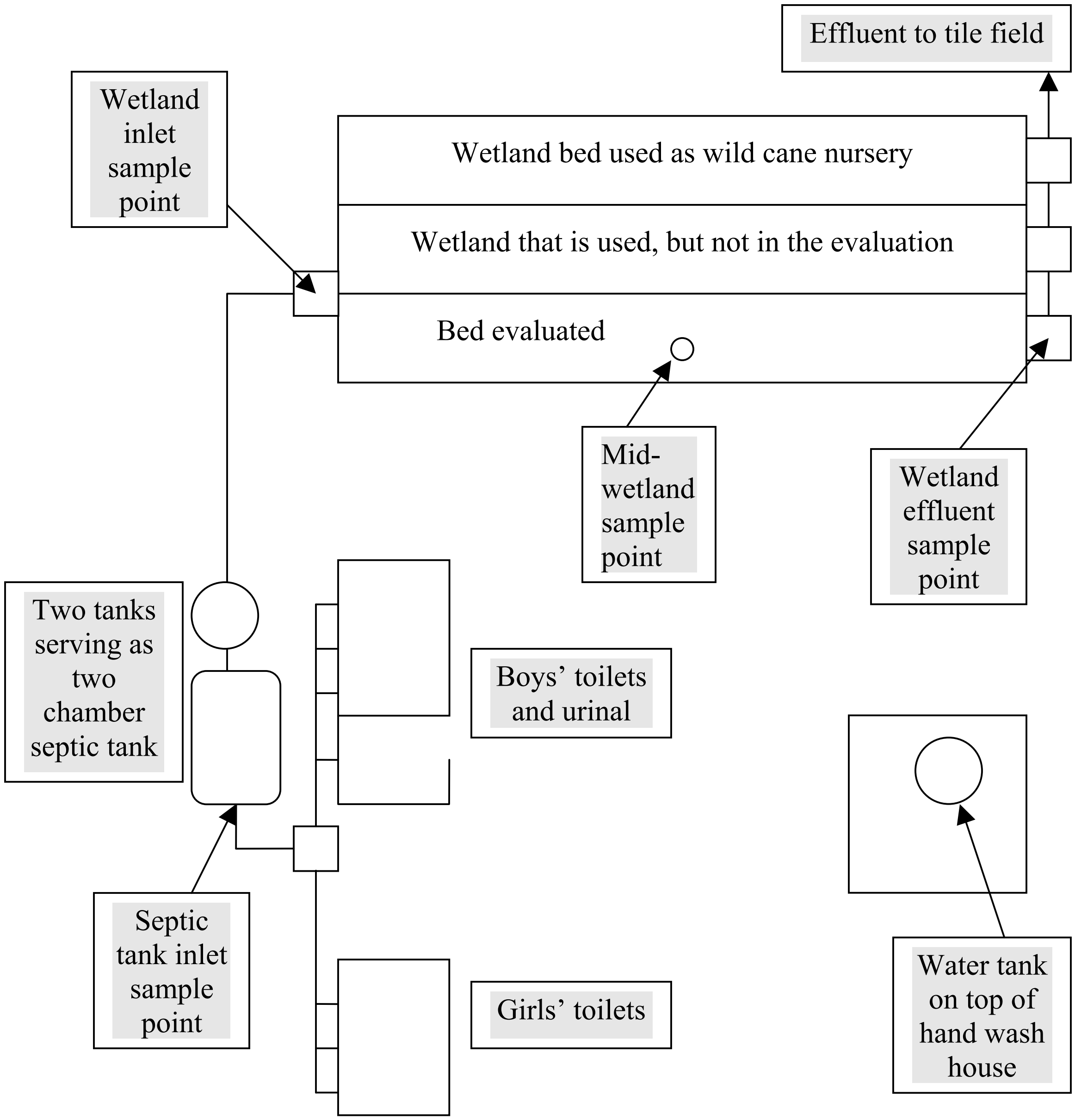

Figure 3-6 Schematic of Retrieve sanitation system showing sampling locations.

Figure 3-7 Retrieve youth in front of boys' toilet building. The concrete structure to the left is the open air urinal. (Photograph by author.)

All toilets flush through 4-inch PVC pipe into one horizontal 880-US gallon plastic tank that empties into a vertical 400-gallon plastic tank. This follows the same design as the Pisgah septic tanks. This system was built with one snorkel pump aerator in the 800-gallon tank that was powered by two solar panels. The aerator was disconnected several weeks prior to the evaluation. The septic tank effluent flows approximately 50 feet horizontally with slight downward pitch to the inlet distribution manhole for the wetlands.

Figure 3-8 Retrieve wetland at the beginning of the evaluation with two year old wild cane. (Photograph by author.)

The three parallel wetlands beds are all concrete with inside dimensions of 47.5ft (14.5m) length x 7.3ft (2.2m) width x 1.6ft (0.5m) depth of media. Each bed has a surface area of 347 ft^2 (32.2 m^2) with L:W aspect ratio of 6.5:1. Since the media is identical to the rough, washed stone at Pisgah, the porosity should be the same neglecting plant root contributions. Two of the three beds have been in use and have mature wild cane (see Figure 3-8) that was planted in September 2002. The third bed is the wild cane nursery for other treatment wetlands. Effluent empties into a small tile field.

Chapter 4 - Methodology

Field Data Collection

Site evaluation began on August 17, 2004 prior to the beginning of the school term to gather background data that would reveal the status of the systems under no loading. The field visits continued over a seventeen-week period ending on December 14, 2004 as the school term finished. Both sites were visited a total of sixteen times to collect water quality samples and record operational data.

Water quality samples included one set of background, five sets of in-service samples and one duplicate set for inter-laboratory comparison. Water quality grab samples were collected from three points along the treatment process to measure the progress through each unit operation. Sample point locations are indicated in Figure 3-3 and Figure 3-6. The first sample at each site was drawn from the top of the first septic tank (first chamber) to represent the raw sewage influent. The second sample was drawn from the wetland inlet distribution manhole or inlet distribution pipe. The third sample was to represent the effluent. At Retrieve, this sample was collected at the wetland outlet. Because there was never effluent at Pisgah, a surrogate sample was collected near the middle of the wetland at the edge of the flourishing, tall wild cane. This mid-wetland sample point was a slotted 4-inch PVC pipe reaching the bottom of the bed. Water from this point should have passed through the most significant portion of the treatment process. All samples with the exception of the effluent at Retrieve were collecting using a basting syringe (see Figure 4-1).

Figure 4-1 An example of grab sample collection. The author is using a syringe to collect the wetland inlet water quality sample at Retrieve. (Photograph of author.)

The effluent sample at Retrieve was collected by boring a hole in the bottom of the wetland effluent pipe and attaching a two-liter plastic bottle to collect water as it trickled out (see Figure 4-2). Locating the hole at the bottom of the pipe may have resulted in a higher concentration of solids in the sample bottle that what exited the wetland. On October 5 the two-liter bottle was not sufficient to fill all the jars for the NWC laboratory and for inter-laboratory comparison. In this instance, water was added from the mid-wetland sample point to make one homogenous sample of sufficient volume.

Figure 4-2 Retrieve effluent collection apparatus showing pipe tap and collection bottle under a steel grill. (Photograph by author.)

One duplicate sample was collected for each round, making a total of seven samples for each sample set taken to the NWC laboratory in Bogue, St. James (Montego Bay metropolitan area). Each BOD, TSS, nutrient and pH sample was stored in an acid washed one half gallon HDPE jug. Each coliform sample was stored in a sterilized 250-ml glass bottle. All samples were transported at ambient temperature as is customary for samples delivered to the NWC laboratory within three hours of collection.

The NWC lab was not provided a duplicate when inter-laboratory comparative samples were collected. On this occasion, six samples went to NWC and seven samples went to the National Environment and Planning Agency (NEPA) laboratory in Kingston. The NEPA BOD samples were collected in BOD jars. Each TSS/nutrient sample for NEPA was placed in a 500-ml plastic bottle and the coliform samples were collected in 250-ml glass bottles. The NEPA samples collected on October 4 were transported on ice and tested the following day.

Data were gathered each week to assess the operating conditions. Temperature of the water at the wetland inlet and wetland midpoint was measured using a mercury thermometer. Approximate height and condition of the wetland grass was recorded. Student population was gathered each week from the respective school offices. Date and time of each visit was recorded to facilitate later investigations in case such data could be helpful.

Data were collected to determine ET rate according to the following basic water balance formula: ET = Inflow + Precipitation - Infiltration $\pm$ Storage – Outflow. Inflow was determined by mechanical water meters supplied by NWC. These were installed on the water supply to the toilets. Accuracy of the meters was confirmed using a 5-gallon bucket. It was assumed that there was no leakage, no infiltration, and the users contributed no significant amount of liquid volume to the wastewater. Accumulated precipitation was measured weekly using rain gages at each site. Storage was determined by measuring the level of water in each wetland at the mid-wetland sample points using a rod and tape measure. There were no flow meters to record outflow so ET calculation was only possible when outflow was zero. The Pisgah system always had zero outflow. The Retrieve system had an unknown volume of outflow every week except for two weeks when the water supply was cut off.

Figure 4-3 Media porosity measurement using a 5-gallon bucket at Pisgah. (Photograph by author.)

Media porosity was measured by filling a bucket with rocks and measuring the quantity of water required to fill the voids to the confirmed 5-gallon mark (see Figure 4-3). This did not take into account any changes resulting from root growth which in one study was estimated to fill 2.5 – 7.9% of the void volume (George et al., 2000).

One alkalinity sample was collected for each source and tested at NWC. Septic tank sludge depth was measured at the end of the evaluation using dip sticks with white cloth wrapped around the end (see Figure 4-4). Since the sandy sludge did not want to stick to the white cloth, the depth was estimated by sensing when the tip of the dip-stick touched the top of the sludge mound.

Figure 4-4 Determining septic tank sludge depth using dip stick at Pisgah. (Photograph of author.)

Laboratory Water Quality Analysis

All tests were conducted at NWC laboratory in Bogue according to the analytical methods listed in Table 4-1.

Table 4-1 Laboratory Test Methods Used in this Evaluation

TEST	METHOD	REFERENCE
BOD	Standard Method 5210B Unseeded Five Day Test. Method 4500-O C Azide Modification for Dissolved Oxygen	APHA, 1998
TSS	Standard Method 2540D	APHA, 1998
Total Nitrogen	Potassium Persulfate Digestion followed by Brucine Method 419D	D'Ella et al., 1977 APHA, 1975
Nitrate	Brucine Method 419D	APHA, 1975
Ammonia	Hach Salicylate Method 10031	Hach, 2002
Total Phosphorus	Standard Method 4500-P B Persulphate Digesion followed by 4500-P D Stannous Chloride	APHA, 1998
Total Coliform	Standard Method 9221B Presumptive and Confirmatory	APHA, 1998
Fecal Coliform	Standard Method 9221B Presumptive and 9221E Confirmatory w/ EC media	APHA, 1998

Immediately upon arrival at the NWC lab, a portion of each bulk sample was transferred to 300-ml BOD jars for the unseeded five-day BOD test (see Figure 4-5). Multiple tube fermentation coliform presumptive tests also began the same day. Unused grab samples remained in their original container and were placed in the BOD incubator/refrigerator. Nutrient tests were usually performed one day after sample collection. Total nitrogen was measured by a Potassium Persulfate digestion method (D'Ella et al., 1977) which converts all nitrogen to nitrates. All nitrate tests followed the Brucine Method 419 D

(APHA, 1975). Ammonia was measured using Hach Test 'N Tube Vials according to Hach Salicylate Method 10031 which is for a concentration range of 0.4 – 50.0 mg/l NH_3-N. This method converts all ammonia to ammonium so that both are measured. Total phosphorus was determined using Persulphate Digestion Method 4500-P B followed by the Stannous Chloride Method 4500-P D. An Orion model 230A was used for pH determination. Rejection of outlier data was performed according to Standard Method 1010 B (APHA, 1998).

Figure 4-5 Dissolved oxygen titration for BOD test. (Photograph by author.)

The inter-laboratory comparison of one set of samples indicated that the variation between lab test results for TSS, nitrate, pH and the low range of fecal coliform were within the range of test method variation. The (NEPA) laboratory does not test fecal coliform to completion so high concentrations of fecal coliform were not compared. The

NEPA laboratory also does not test total coliform or ammonia and was not yet testing for total nitrogen at the time of this study. The BOD and phosphorus inter-laboratory comparison test results indicated a discrepancy between the two laboratories. Further investigation will be conducted by NWC.

Chapter 5 – Results and Discussion

Pisgah Sanitation System Hydraulics

The volumetric flowrate of the water supplying the toilets ranged from 880 liter/wk to as high as 7,930 liter/wk when a few toilet flappers remained suspended causing water to flow constantly. Average water flow was 1,850 liter/wk or 132 liter/day (35 USgal/day) to each septic tank. Averaged over the entire week this is 1.3 liters per student per day. Hydraulic retention time in the boys' and girls' septic tanks averaged 29 days assuming half of the flow went to each tank.

The average combined hydraulic loading from sewage and precipitation for the evaluation period was 0.25 USgal/ft^2-d (76 L/m^2-d). Hydraulic loading of the wetland from sewage alone ranged from 0.03 – 0.09 USgal/ft^2-d (9.1 – 27 L/m^2-d) during normal operation and as high as 0.31 USgal/ft^2-d (94 L/m^2-d) when the toilets were constantly running. The water in the wetland never was high enough to generate effluent even during this area's typical heavy rainfall of 3.3 inches (84 mm) to 5.3 inches (135 mm) per week. The water level in the wetland ranged from 0.5 inch (13 mm) to 9.3 inches (236 mm) which caused the plant roots to extend deeply into the media in search of water. During installation of the mid-wetland sampling point it was noted that plant roots were thin but reached to within three inches of the bed bottom.

The evapotranspiration (ET) rate for the constructed SSF wetland at Pisgah All Age School was determined by averaging ten weeks of ET data. The average ET rate was determined to be 0.27 USgal/ft^2-d (equivalent to 11 mm/day with 100% porosity). This is slightly higher than the recommended design rate of 0.26 USgal/ft^2-d for ET beds in Miami, Florida (Bernhart, 1972) and slightly higher than the 1.5 to 2 times pan evaporation reported for SSF wetlands in the United States (USEPA, 2000). The National Meteorological Service pan evaporation data from 2001 - 2003 at Sangster Airport in Montego Bay averaged 4.9 mm/day.

Since evapotranspiration decreases fluid content without removing salts, there was concern regarding excessive increase in the concentration of total dissolved solids (TDS) over time. Water samples collected in the wetland after a year and a half of service had conductance ranging from 438 – 447 μmho (see Appendix 4). This range is comparable to the conductance measurements for the Retrieve wetland and below the regularly measured levels found in the effluent from oxidation ditches and stabilization ponds in Western Jamaica (NWC, 2004). Insufficient data was collected to make a conclusive determination regarding TDS buildup, but there was no indication that TDS accumulation was excessive.

Retrieve Sanitation System Hydraulics

The volumetric flowrate of water to the toilets and urinal ranged from 10,540 liter/wk to 49,090 liter/wk. Average water use for toilets and urinal was 22,680 liter/wk or 3,240 L/d (857 USgal/day). Averaged over the seven day week this is 48.3 L/p-d. Hydraulic retention time in the septic tank averaged 1.2 days. The constant urinal wash water and occasional malfunction of toilet flappers contributed to the high water use and resultant low septic tank HRT. This school pays a flat rate water bill.

Hydraulic loading on the wetland from sewage ranged from 1.1 – 5.3 USgal/ft^2-d (45 - 216 L/m^2-d). The heaviest weekly precipitation accumulation at this site was 3.2 inches (81 mm). Hydraulic loading on the wetland from sewage and rainfall combined ranged from 1.4 – 5.3 USgal/ft^2-d (57 – 216 L/m^2-d) with an average of 2.6 USgal/ft^2-d (105 L/m^2-d). Average HRT for the single wetland bed used during the evaluation was 2.2 days.

On two occasions, high influent flow rate caused surface flooding (i. e. water over the rock surface) at the inlet of the wetland and near overflow water level at the inlet manhole. This was observed when average influent flow and precipitation exceeded 5,055 L/d (1,335 USgal/d) and 7,010 L/d (1,852 USgal/d) corresponding to hydraulic loading of 3.85 USgal/ft2-d (147 L/m^2-d) and 5.34 USgal/ft^2-d (204 L/m^2-d). Insufficient hydraulic conductivity (i. e. high resistance to horizontal flow through the media) was the cause of surface flooding. The 6.5:1 length to width ratio of this wetland bed reduces the possibility of short circuiting so that treatment is improved but decreases hydraulic conductivity which will result in surface flooding when maximum flow capacity is reached. The maximum flow capacity was not determined. The outlet water level could have been lowered to reduce or eliminate the flooding, but no changes were made in order to keep conditions constant through the evaluation. The high water level in the manhole may have been caused by partial plugging of the slits in the inlet distribution pipe or solids accumulation in the media at the inlet zone. During installation of the mid-wetland sample pipe the plant roots were observed to be a thick mat in the top 4 inches of the media and did not significantly reach beyond this depth. This root mat could have created resistance to surface flooding thus forcing the water in the manhole to back up until sufficient head pressure was available to overcome horizontal flow resistance.

The surface flooding events provide some useful data for estimating the hydraulic conductivity of the front end of the wetland. Darcy's law is meant to represent flow through a porous media and is used in wetland design to determine head loss through the media based on an assumed hydraulic conductivity. Darcy's law is written as follows: Q= K Ac S where Q = flow rate (m^3/d); K = hydraulic conductivity (m^3/m^2-d); Ac = cross sectional area (m^2); and S = hydraulic slope gradient (m/m). The cross sectional area Ac = W Dw where W = width of wetland and Dw = average depth of water. The hydraulic slope S = dh/dL where dh = change in water height (i. e. head loss) and dL = length of travel through wetland corresponding to dh. This results in the following formula: Q = K W Dw dh/dL or K = Q dL / dh W Dw.

The lowest flow rate that seemed to create surface flooding was 4.73 m^3/d which when combined with precipitation was 5.055 m^3/d. The length of wetland travel was half the full length of 47.5 ft (14.48 m) or 7.24 m. The manhole elevation is approximately 2 inches higher than the 24-inch high wetland walls that contain 20 inches of rock media.

The water level in the middle of the wetland was 19.5 inches. Assuming there was no overflow at the manhole, the water level in the manhole was 6.5 inches (0.165m) higher than in the middle of the wetland bed. The width of the wetland is 7.3 ft or 2.23 m. Using wetland bottom as reference, the average depth of water between the manhole and mid-wetland point is 26 inches (0.660 m) minus 19.5 in (0.495 m) or 0.576 m. This data may be summarized as Q = 4.73 m^3/d, dL = 7.24 m, dh = 0.165 m, W = 2.23 m, and Dw = 0.576 m. Applying Darcy's Law equation K = Q dL / dh W Dw results in the hydraulic conductivity K = 172 m^3/m^2-d.

This is a reasonable estimate for the average hydraulic conductivity of the first 50% of a wetland bed that has been in service for two years. The field measured porosity of 37.7% corresponds to a hydraulic conductivity (K) in the range of 1,000 to 50,000 m^3/m^2-d (Reed, 2001). It is recommended that 1% of the clean conductivity be used for the initial 30% of the bed and the remaining 70% of the bed is assumed to have a K of 10% of the clean K value (USEPA, 2000). If it is assumed that a clean K equals 10,000 m/d, then the first 30% of the bed would have a K of 100 m/d and the remainder of the bed K would have a K of 3,000 m/d.

Since wetland had an unmeasured quantity of effluent every week except for two weeks when the water supply was unavailable and the outflow was zero, the ET rate was based on data from these two weeks. During the water outage, buckets were used to supply a minimal amount of water for flushing so precipitation was the major source of water entering the system. ET based upon these two weeks was an estimated 0.24 USgal/ft2-d or10 mm/day. Under normal operating conditions the water level is high in this wetland so the water reaches the plant roots. During the two weeks when the water supply was off, the water input was so low that the water level dropped below the root zone which may have caused a lower than normal ET rate.

Water Quality Test Results

Biochemical Oxygen Demand (BOD)

BOD is mostly reduced in a septic tank by primary settling of organic solids. Particulate matter is removed in SSF wetlands through flocculation and sedimentation, discrete settling, filtration and adsorption onto biofilm. The organic particles eventually dissolve or are broken up into smaller particles that might exit as colloids that will increase the BOD and TSS measured in the effluent (USEPA, 2000). Soluble BOD is consumed biologically at a rate that is temperature dependent. Water temperature in both of the wetlands remained a relatively warm 22 - 26°C.

The results for five-day BOD tests are presented in Table 5-1. Wastewater entered the Pisgah septic tanks with an average BOD of 58 mg/l and septic tank effluent entered the wetland with an average BOD of 27 mg/l. The Pisgah mid-wetland samples averaged 13 mg/l BOD. Wastewater entered the Retrieve septic tanks with average BOD of 40 mg/l, then entered the wetland with an average BOD of 15 mg/l and left the wetland with an average BOD of 20 mg/l. The average wetland BOD mass loading was 0.79 kg/ha-d (0.71 lb/ac-d) at Pisgah and 15 kg/ha-d (13 lb/ac-d) at Retrieve. These were well below the USEPA recommended maximum of 60 kg/ha-d (USEPA, 2000).

The 5-mg/l increase in BOD across the Retrieve wetland could have been a combination of the effluent sample collection method and background contributions from plant and microbe decay. Wetlands produce a low level of background BOD so that zero BOD effluent is not possible but may approach 10 mg/l (Reed et al., 2001). The potential problems associated with the effluent sample method were discussed in the previous report section on TSS.

An overall BOD reduction of 78% was measured for the Pisgah system and an overall reduction of 50% was measured for the Retrieve system. Septic tank removal of influent

BOD was determined to be 53% at Pisgah and 63% at Retrieve. These are within the typical BOD removal efficiency range of 46 – 68% reported in literature (Seabloom et al., 1982; Rahman et al., 1999). The Pisgah wetland removed 52% of its influent BOD. This was below the 68.5 – 92.7% removal efficiency reported in literature for SSF wetlands possibly because of the low influent concentration (Vymazal, 2002).

Table 5-1 Five-Day Biochemical Oxygen Demand for Pisgah and Retrieve Sanitation Systems

Sample	Arithmetic Mean ($\overline{X}$) mg/l	90% Confidence Interval mg/l	Sample Size (n)
Pisgah First Septic Tank	58	$\overline{X} \pm 20$	5
Pisgah Wetland Influent	27	$\overline{X} \pm 9$	4
Pisgah Wetland Effluent	13	$\overline{X} \pm 5$	3
Retrieve First Septic Tank	40	$\overline{X} \pm 24$	6
Retrieve Wetland Influent	15	$\overline{X} \pm 10$	5
Retrieve Wetland Effluent	20	$\overline{X} \pm 7$	7

The first two sets of water quality samples indicated that the water in the top of the first chamber of the septic tanks had BOD ranging from 8 to 69 mg/l which was lower than expected. Typical BOD for raw sewage ranges from 110 – 350 mg/l in the United States (Tchobanoglous et al., 2003). Influent BOD to the Montego Bay treatment ponds and the oxidation ditches in Ocho Rios ranges from 47 – 555 mg/l (NWC, 2005). Because of the low influent BOD values, the collection point for subsequent samples was moved from the center of the first tank to the inlet tee of the first tank. Although there were some fecal particles in subsequent samples, the BOD of septic tank influent ranged from 24 to 83 mg/l.

The temperature for five-day incubation of BOD samples is to remain 20ºC (68ºF) ± 1ºC. However, the temperature in the NWC incubator from August 17 to November 15 ranged from 14.0 - 17.5ºC. After repairs the temperature ranged between 18.5 - 20.5ºC from

November 19 to December 8. Lower temperatures reduce oxygen consumption from biological activity so that BOD samples incubated below 20ºC have slightly lower BOD.

The full set of comparison samples sent to NEPA on October 5 had to be stored on ice overnight. It was discovered too late that water from melting ice had removed the taped labels on the BOD jars. The NEPA laboratory BOD values for these samples ranged from 16 to 135 mg/l compared to NWC laboratory BOD values that ranged from 11 to 69 mg/l. A second inter-laboratory comparison of BOD had similar results and indicated that sample handling was not the source of discrepancy. Further investigation to determine why NWC laboratory BOD values are lower than NEPA laboratory values will be conducted by NWC.

Total Suspended Solids (TSS)

The results for the TSS tests are presented in Table 5-2. Wastewater entered the Pisgah septic tanks with an average TSS of 92 mg/l and septic tank effluent entered the wetland with 57 mg/l TSS. The Pisgah mid-wetland sample had an average of 13 mg/l TSS. Wastewater entered the Retrieve septic tank with an average TSS of 41 mg/l, entered the wetland with 6 mg/l and then left the wetland with an average TSS of 98 mg/l. The average wetland TSS mass loading was determined to be 0.17 kg/ha-d (0.15 lb/ac-d) at Pisgah and 0.64 kg/ha-d (0.57 lb/ac-d) at Retrieve. This was well below the USEPA recommended maximum of 200 kg/ha-d (USEPA, 2000).

The 92 mg/l increase in suspended solids measured for water passing through the Retrieve wetland may have been caused by capturing the sample from the bottom of the effluent pipe. The piping design prevented collecting a grab sample in the same manner as the other sample points, and the low effluent flow rate did not allow filling the empty bottles by hand within a reasonable amount of time. To obtain sufficient sample volume, a two-liter plastic bottle was connected to the bottom of the effluent pipe allowing water to trickle in over time. Pulling from the bottom of the pipe would tend to collect more solids than a sample point that pulled from the top or mid-stream of flow, and it would allow non-buoyant solids to continue to enter the bottle even when full of liquid. Another contributor to high solids may have been the atmospheric exposure of the effluent prior to sample capture. On some occasions there were water bugs or mosquito larvae in the sample bottle.

Table 5-2 Total Suspended Solids for Pisgah and Retrieve Sanitation Systems

Sample	Arithmetic Mean ($\overline{X}$) mg/l	90% Confidence Interval mg/l	Sample Size (n)
Pisgah First Septic Tank	92	$\overline{X} \pm 44$	5
Pisgah Wetland Influent	57	$\overline{X} \pm 57$	5
Pisgah Wetland Effluent	13	$\overline{X} \pm 9$	4
Retrieve First Septic Tank	41	$\overline{X} \pm 21$	6
Retrieve Wetland Influent	6	$\overline{X} \pm 4$	5
Retrieve Wetland Effluent	98	$\overline{X} \pm 67$	7

An overall TSS reduction of 85% was measured in the Pisgah system but an overall increase in TSS was measured at Retrieve. The Pisgah septic tanks removed an average of 38% of their influent and the Retrieve septic tank removed 85%. These are near the removal efficiencies of 30 – 81% expected for septic tanks (Seabloom et al., 1981; Rahman et al., 1999). The Pisgah wetland removed 77% of influent TSS. This was within the 63.0 – 89.8% removal efficiency range for SSF wetlands reported in literature (Vymazal, 2000).

Entry zone solids loading rates for both wetlands averaged well below the recommended design limit of 0.008 lb/ft2-d (39 g/m2-d) (Crites et al., 1998). The average loading was 0.0013 lb/ft2-d (6.4 g/m2-d) at Pisgah and 0.0038 lb/ft2-d (19 g/m2-d) at Retrieve. Entry zone loading reached an estimated maximum of 0.0081 lb/ft2-d (40 g/m2-d) during the week of highest flow rate at Retrieve.

Total Nitrogen, Ammonia and Nitrate

The results for the total nitrogen and ammonia tests are presented in Tables 5-3 and 5-4. Wastewater entered the Pisgah septic tanks with an average of 33.3 mg/l total nitrogen and 76 mg/l ammonia nitrogen. Water entered the wetland with 39.8 mg/l total nitrogen and 58 mg/l ammonia nitrogen. This indicates some error in testing since ammonia should be included in the total nitrogen assay. The two factors that may have contributed to this anomaly were the ammonia test method upper limit of 50 mg/l was exceeded by three of the five septic tank influent samples from Pisgah, and testing for total nitrogen was a new procedure for the laboratory. Water from the mid-wetland sample point at Pisgah averaged 1.6 mg/l total nitrogen and 0.4 mg/l ammonia. Wastewater entered the Retrieve septic tanks with an average to 20.0 mg/l total nitrogen and 13 mg/l ammonia. The water entered the wetland with 6.8 mg/l total nitrogen and 5.8 mg/l ammonia. Effluent left the wetland with 7.8 mg/l total nitrogen and 0.4 mg/l ammonia. The 1.0 mg/l increase in total nitrogen measured for the water passing through the Retrieve wetland may have been caused by high solids resulting from the effluent sample collection method.

Table 5-3 Total Nitrogen for Pisgah and Retrieve Sanitation Systems

Sample	Arithmetic Mean ($\overline{X}$) mg/l	90% Confidence Interval mg/l	Sample Size (n)
Pisgah First Septic Tank	33.3	$\overline{X} \pm 18.4$	5
Pisgah Wetland Influent	39.8	$\overline{X} \pm 15.9$	5
Pisgah Wetland Effluent	1.6	$\overline{X} \pm 1.0$	4
Retrieve First Septic Tank	20.0	$\overline{X} \pm 15.1$	6
Retrieve Wetland Influent	6.8	$\overline{X} \pm 2.2$	6
Retrieve Wetland Effluent	7.8	$\overline{X} \pm 5.2$	7

Table 5-4 Ammonia (NH_3-N) for Pisgah and Retrieve Sanitation Systems

Sample	Arithmetic Mean ($\overline{X}$) mg/l	90% Confidence Interval mg/l	Sample Size (n)
Pisgah First Septic Tank	76	$\overline{X} \pm 49$	5
Pisgah Wetland Influent	58	$\overline{X} \pm 39$	5
Pisgah Wetland Effluent	0.4	$\overline{X} \pm 0.5$	4
Retrieve First Septic Tank	13.0	$\overline{X} \pm 11$	6
Retrieve Wetland Influent	5.8	$\overline{X} \pm 2.2$	6
Retrieve Wetland Effluent	0.4	$\overline{X} \pm 0.5$	7

It was determined that the Pisgah sanitation system removed an average of 95% of the total nitrogen and 99% of the ammonia from the wastewater. The Retrieve sanitation system removed an average of 68% of the total nitrogen and 97% of the ammonia. The Pisgah septic tanks removed 11% of influent total nitrogen and 24% of influent ammonia. The Retrieve septic tank removed 66% of influent total nitrogen and 55% of influent ammonia. It was determined that the Pisgah wetland removed 95% of its influent total nitrogen. This was above the 60.1 – 64.8% removal efficiencies reported for horizontal SSF wetlands (Vymazal, 2002). It was determined that influent concentration of ammonia was reduced by 99% in the Pisgah wetland and 93% in the Retrieve wetland. This was above the 64% average ammonia removal efficiency reported for ten horizontal SSF wetlands in Italy (Masi et al., 2000).

At Pisgah the average total nitrogen and ammonia loads entering the wetland were 0.87 kg/ha-d N (0.78 lb/ac-d) and 1.69 kg/ha-d NH_3-N (1.51 lb/ac-d). At Retrieve the average loads entering the wetland were 6.8 k/ha-d N (6.1 lb/ac-d) and 0.17 kg/ha-d NH_3-N (0.15 lb/ac-d). Total nitrogen removal in the wetland at Pisgah was 0.83 kg/ha-d (0.74 lb/ac-d) and ammonia nitrogen removal was 1.68 kg/ha-d (1.50 lb/ac-d). At Retrieve the data indicated an increase of total nitrogen through the wetland and ammonia removal of 0.16 kg/ha-d (0.14 lb/ac-d). The high ammonia removal compared to total nitrogen removal in both wetlands could indicate error in the test results and the addition of particulate organic nitrogen simultaneous to the removal of soluble ammonia. Plant senescence could have produced organic nitrogen while plant growth absorbed ammonia. In terms of influent concentrations, the removal efficiencies were high for total nitrogen and ammonia at the Pisgah wetland and high for ammonia removal at the Retrieve wetland. However, the removal of total nitrogen and ammonia in terms of areal mass loading was within the values reported in literature. It has been reported that nitrogen removal from plants can reach as high as 1.6 kg/ha-d (1.4 lb/ac-d) if regular harvesting is performed to prevent dying plants from releasing nutrients back into the water (EPA, 2000). Optimized pilot scale harvesting of cattail was found to be every 8 weeks and resulted in total nitrogen removal of 7.1 kg/ha-d (6.3 lb/ac-d) with wetland HRT of 5 days (Koottatep et al., 1997).

The nitrate test results are presented in Table 5-5. Average nitrate concentrations in the Pisgah sanitation system were measured to be 0.2 mg/l entering the septic tanks, 0.5 mg/l exiting the septic tanks and 0.7 mg/l at the mid-wetland sample point. Average nitrate concentrations in the Retrieve sanitation system were measured to be 0.4 mg/l entering the septic tank, 0.4 mg/l exiting the septic tank and 0.5 mg/l exiting the wetland. These low nitrate levels indicate a normal functioning system. Nitrogen from septic tank effluent is typically more ammonia than organic nitrogen and little or no nitrate (Crites, et al., 1998). SSF wetlands produce nitrate only when there has been biological nitrification in the wetland and insufficient carbon for the heterotrophic bacteria to convert the nitrate to nitrogen gas (George, et al., 2000).

The organic nitrogen particles entering a wetland are trapped in the wetland media and converted by bacteria to ammonia (Reed, 2001; Tchobanoglous et al., 2003). Ammonia in the wastewater entering a wetland may be removed by volatization to the atmosphere, plant uptake, adsorption by the media or biological nitrification and denitrification.

Table 5-5 Nitrate (NO_3-N) for Pisgah and Retrieve Sanitation Systems

Sample	Arithmetic Mean ($\overline{X}$) mg/l	90% Confidence Interval mg/l	Sample Size (n)
Pisgah First Septic Tank	0.2	$\overline{X} \pm 0.1$	5
Pisgah Wetland Influent	0.5	$\overline{X} \pm 0.3$	5
Pisgah Wetland Effluent	0.7	$\overline{X} \pm 0.4$	4
Retrieve First Septic Tank	0.4	$\overline{X} \pm 0.3$	6
Retrieve Wetland Influent	0.4	$\overline{X} \pm 0.3$	6
Retrieve Wetland Effluent	0.5	$\overline{X} \pm 0.3$	6

Unionized ammonia (NH_3) is toxic to many fish (Reed et al., 1995) and can evaporate out of wastewater whereas ammonium (NH_4^+) cannot. The low pH found in a wetland causes ammonium to be the predominant species so that very little unionized ammonia is available to be removed by volatilization. The percentage of ammonia in the unionized form may be determined by the chemical equilibrium equation $\frac{[\mathrm{NH_3}][\mathrm{H^+}]}{[\mathrm{NH_4^+}]} = K$ where K is the equilibrium constant. At pH 9.25 the NH_3 and NH_4^+ species are in equilibrium so that $K = 10^{-9.25}$. At the typical wetland pH of 7.5, $[H^+] = 10^{-7.5}$ so that the percentage of ammonia (NH_3) equals $\frac{[\mathrm{NH_3}]\times 100}{[\mathrm{NH_3}]+[\mathrm{NH_4^+}]} = \frac{100}{1+[\mathrm{H^+}]/K} = \frac{100}{1+\left(10^{-7.5}/10^{-9.25}\right)} = 1.8\%$

Ammonia and ammonium removal by plant uptake is more rapid with growing plants than mature plants (Koottatep et al., 1997; Reed, 2001). Ammonia removal will reach steady-state once plant density reaches maximum and the cycle of plant growth and death equalizes. Media adsorption will contribute to ammonia removal until all the absorption sites are saturated (Reed et al., 2001).

Biological nitrification and denitrification has been reported to be the primary removal mechanism for nitrogen in SSF wetlands (Crites et al., 1998; Reed et al., 2001).

Evidence from 14 horizontal flow SSF wetlands operating in the United States indicated deep roots and long HRT improved ammonia removal through nitrification and denitrification (EPA, 1993). Ammonium removal in pilot scale SSF wetlands was found to be better in beds planted with softstem bulrush than beds without plants (George, et al., 2000). This evidence supports the root zone concept of sufficient oxygen for biological nitrification being supplied to the wastewater by the roots of the aquatic plants. Another report stated that full-scale SSF wetlands built for ammonia removal performed below expectations and some other technology besides the single pass horizontal SSF wetland is required if ammonia removal is critical. It was concluded that plants cannot contribute a significant amount of oxygen to a SSF wetland (EPA, 2000).

Nitrification is often the limiting step in converting ammonia to nitrogen gas because of the low concentration of dissolved oxygen (Reed et al., 1995). Biological nitrification occurs under optimal conditions for growth and sustenance of the aerobic autotrophic nitrifying bacteria. These conditions include the following:

- Carbon Source

  A BOD of at least 15 – 20 mg/l is recommended (Reed et al., 2001).

  Too much BOD will result in competition with heterotrophic bacteria (Tchobanoglous et al., 2003).
- Temperature

  Optimum temperature is 30 – 35ºC with little nitrification occurring below 5ºC or above 40ºC (Hammer et al., 1994). Microbes will work twice as fast at 24ºC compared to 12ºC.
- pH

  Optimal range is 7.5 to 8.0 but reasonable nitrification can occur at 7.0 (Tchobanoglous et al., 2003).
- Dissolved Oxygen (DO)

  Rate of nitrification is reduced at DO concentrations below 2 mg/l (Hammer et al., 1994) and the conversion of nitrite to nitrate is greatly inhibited at DO concentrations below 0.5 mg/l (Tchobanoglous et al., 2003).

- Alkalinity

  Nitrification and denitrification of 1 g NH_3-N requires a net 3.58 g of alkalinity as $CaCO_3$ (See below).

Table 5-6 lists the favorable range of the parameters influencing biological nitrification and the test results for the Pisgah and Retrieve sanitation system wetlands. The carbon source availability in the wetlands as indicated by the BOD concentrations entering and exiting the wetlands ranged from 9 – 38 mg/l at Pisgah and 9 – 32 mg/l at Retrieve. These were close to the recommended range of 15 – 20 mg/l (Reed et al., 2001). The wetland water temperatures of 22 – 26ºC at Pisgah and 23 – 26ºC at Retrieve were below the optimum temperature range but within the necessary range of 5 - 40ºC (Hammer et al., 1994). The pH ranges of 6.7 -7.9 at Pisgah and 6.7 – 7.8 at Retrieve indicate the pH sometimes fell below 7.0 where reasonable nitrification can occur, but was normally near the optimum range of 7.5 – 8.0 (Tchobanoglous et al., 2003). Insufficient DO data was gathered to make a conclusive comment. Alkalinity of the rainwater at Pisgah was measured to be 8.1 mg/l as $CaCO_3$. Alkalinity of the municipal water supply at Retrieve was measured to be 210 mg/l as $CaCO_3$. Alkalinity can be shown to be a limiting factor for biological nitrification at Pisgah.

Table 5-6 Parameters Influencing Biological Nitrification and the Measured Values for Pisgah and Retrieve Wetlands. The alkalinity ratio applies to systems in which biological denitrification also occurs.

Parameter	Optimum or Recommended Range	Measured Range Pisgah	Retrieve
BOD	15 – 20 mg/l	9 – 38 mg/l	6 – 32 mg/l
Temperature	30 - 35ºC	22 - 26ºC	23 - 26ºC
pH	7.5 – 8.0	6.7 – 7.9	6.7 – 7.8
DO	> 2.0 mg/l	n/a	n/a
Alkalinity	> 3.58g $CaCO_3$: 1g NH_4-N	0.14 : 1	36 : 1

Biological Nitrification Reaction (Crites et al., 1998).

$$NH_4^+ + 1.731O_2 + 1.962HCO_3^- \rightarrow 0.038C_5H_7NO_2 + 0.962NO_3^- + 1.077H_2O + 1.769H_2CO_3$$

Biological Denitrification Reaction (Crites et al., 1998).

$$NO_3^- + 1.183CH_3OH + 0.273H_2CO_3 \rightarrow 0.091C_5H_7NO_2 + 0.454N_2 + 1.820H_2O + HCO_3^-$$

Biologically nitrifying 1.0 g of ammonia nitrogen consumes 7.01 g of alkalinity and denitrifying 1.0 g of nitrate nitrogen produces 3.57 g alkalinity (Crites et al., 1998). Following the above reactions, if 1.0 g of ammonia nitrogen is nitrified then $\frac{3.57\text{g } CaCO_3}{1.0\text{g } NO_3\text{-N}} \times 0.962\text{g } NO_3\text{-N}$ or 3.43 g of alkalinity is produced during denitrification. The 7.01 g of alkalinity consumed in nitrification minus the 3.43 g of alkalinity produced by denitrification yields a net 3.58 grams of alkalinity consumed for every gram of ammonia nitrogen converted to cell tissue and nitrogen gas. Therefore the required mass ratio of alkalinity to ammonia is 3.58:1 assuming all nitrate converts to nitrogen gas. At Pisgah the 8.1 mg/l alkalinity to 58 mg/l ammonia ratio of 0.14:1 is sufficient for converting only 2.3 mg/l of ammonia. This is based on the assumption that the toilet flushing water is the sole source of alkalinity to the system. The Retrieve wetland influent alkalinity to ammonia ratio of 210:5.8 (36:1) is adequate for complete conversion. Therefore the primary mechanism for ammonia removal at Pisgah was not biological nitrification and denitrification but must have been uptake by the wild cane and adsorption by the rock media. Ammonia removal at Retrieve may have been a combination of biological removal, plant uptake and perhaps media absorption.

Total Phosphorus

The results for the total phosphorus tests are presented in Table 5-7. The average total phosphorus concentrations in the Pisgah sanitation system were measured to be 12.8 mg/l entering the septic tanks, 9.6 mg/l entering the wetland and 0.4 mg/l at the mid-wetland sample point. The average total phosphorus concentrations in the Retrieve sanitation system were measured to be 3.3 mg/l entering the septic tanks, 1.5 mg/l entering the wetland and 1.2 mg/l exiting the wetland. The average mass load of phosphorus on the wetlands was 0.28 kg/ha-d (0.25 lb/ac-d) at Pisgah and 1.5 kg/ha-d (1.3 lb/ac-d) at Retrieve.

Average phosphorus removal was determined to be 97% for the Pisgah sanitation system and 64% for the Retrieve system. The septic tank removal efficiencies of 25% at Pisgah and 55% at Retrieve were within the typical phosphate removal efficiency range of 20 – 65% (Seabloom et al., 1982; Rahman et al., 1999). Removal efficiency for wetland influent phosphorus was determined to be 96% at Pisgah and 20% at Retrieve. These results are outside the removal efficiency range of 26.7 – 65.0% reported in literature for SSF wetlands (Vymazal, 2002). The harvest of the wild cane from the Pisgah wetland contributed to the high phosphorus uptake. The high solids accumulation in the Retrieve wetland effluent caused by the sampling method could have contributed to the low phosphorus uptake that was measured.

Table 5-7 Total Phosphorus (P) for Pisgah and Retrieve Sanitation Systems

Sample	Arithmetic Mean ($\overline{X}$) mg/l	90% Confidence Interval mg/l	Sample Size (n)
Pisgah First Septic Tank	12.8	$\overline{X} \pm 7.6$	5
Pisgah Wetland Influent	9.6	$\overline{X} \pm 6.6$	5
Pisgah Wetland Effluent	0.4	$\overline{X} \pm 0.2$	4
Retrieve First Septic Tank	3.3	$\overline{X} \pm 1.6$	6
Retrieve Wetland Influent	1.5	$\overline{X} \pm 0.5$	6
Retrieve Wetland Effluent	1.2	$\overline{X} \pm 0.5$	7

Long term phosphorus removal should not be expected for a wetland unless regular plant harvesting is performed which can yield a maximum of 0.25 kg/ha-d (0.22 lb/ac-d) phosphorus removal (EPA, 2000). The removal of phosphorus was determined to be 0.27 kg/ha-d (0.24 lb/ac-d) at Pisgah and 0.0088 kg/ha-d (0.0079 lb/ac-d) at Retrieve.

Phosphorus entering a wetland is removed primarily by media adsorption and plant uptake. Over time the physical sites for adsorption will become saturated so that only plant uptake will remove phosphorus in a wetland. Rapidly growing plants take up more phosphorus than mature plants so that a mature stand of plants might remove only a moderate amount of phosphorus (Reed et al., 2001).

Total and Fecal Coliform

The results for the total coliform tests are presented in Table 5-8 and the fecal coliform test results are presented in Table 5-9. All averages represent the geometric mean of the measured values. Wastewater entered the Pisgah septic tanks with an average total coliform concentration of 3,500,000 MPN/100ml and a fecal coliform concentration of 3,200,000. The water exited the Pisgah septic tanks with an average total coliform concentration of 422,000 MPN/100ml and fecal coliform concentration of 379,000 MPN/100ml. Water collected at the Pisgah mid-wetland sample point had an average total coliform concentration of 234 MPN/100ml and fecal coliform concentration of 140 MPN/100ml. Wastewater entered the Retrieve septic tank with an average total coliform concentration of 2,040,000 MPN/100ml and a fecal coliform concentration of 3,360,000 MPN/100ml. This average value for fecal coliform concentration is higher than the total coliform because the incubator bath temperature control malfunctioned and spoiled one set of fecal coliform samples that would have lowered the average. Wastewater exited the Retrieve septic tank with an average total coliform concentration of 94,400 MPN/100ml and fecal coliform concentration of 85,000 MPN/100ml. Retrieve wetland effluent had an average total coliform concentration of 386 MPN/100ml and fecal coliform concentration of 73 MPN/100ml.

The treatment efficiency for the Pisgah sanitation system was measured to be a 99.99% reduction in total and fecal coliform. The treatment efficiency for the Retrieve sanitation system was measured to be a 99.98% reduction in total coliform concentration and 99.997% reduction in fecal coliform. The septic tanks at Pisgah reduced total and fecal coliform by 88%. The Retrieve septic tank reduced total coliform by 95% and fecal coliform by 97%. All of these septic tanks performed better than the 25 – 66% coliform removal rates reported in literature for septic tanks (Seabloom et al., 1982; Rahman et al., 1999). The very high coliform reduction in the Retrieve septic tank may have been promoted by dilution from the continuous flow of chlorinated water through the urinal. The Pisgah wetland had a measured 3 log (99.9%) reduction of total and fecal coliform. The Retrieve wetland had a measured 2 log (99%) reduction of total coliform and 3 log reduction of fecal coliform. A 2 to 3 log reduction of fecal coliform is expected for a wetland with 5 – 10 day HRT (Reed et al., 2001). HRT averaged 2.2 days in the Retrieve wetland and 41 days in the Pisgah wetland.

Table 5-8 Total Coliform of Pisgah and Retrieve Sanitation Systems

Sample	Geometric Mean ($\overline{X}$) MPN/100ml	90% Confidence Interval Mean MPN/100ml	Sample Size (n)
Pisgah First Septic Tank	3,500,000	$\overline{X}$ ± 2,780,000	5
Pisgah Wetland Influent	422,000	$\overline{X}$ ± 1,450,000	5
Pisgah Wetland Effluent	234	$\overline{X}$ ± 20,200	4
Retrieve First Septic Tank	2,040,000	$\overline{X}$ ± 11,300,000	6
Retrieve Wetland Influent	94,400	$\overline{X}$ ± 58,700	6
Retrieve Wetland Effluent	386	$\overline{X}$ ± 1,140	7

Table 5-9 Fecal Coliform of Pisgah and Retrieve Sanitation Systems

Sample	Geometric Mean ($\overline{X}$) MPN/100ml	90% Confidence Interval Mean MPN/100ml	Sample Size (n)
Pisgah First Septic Tank	3,200,000	$\overline{X}$ ± 3,280,000	4
Pisgah Wetland Influent	379,000	$\overline{X}$ ± 1,930,000	4
Pisgah Wetland Effluent	140	$\overline{X}$ ± 1,600	3
Retrieve First Septic Tank	3,360,000	$\overline{X}$ ± 14,000,000	5
Retrieve Wetland Influent	85,000	$\overline{X}$ ± 75,100	5
Retrieve Wetland Effluent	73	$\overline{X}$ ± 831	5

pH

The results for the pH tests are presented in Table 5-10. Average pH of the influent to the Pisgah septic system was determined to be 7.3 and influent to the Retrieve septic system was determined to be 7.2. The pH of the effluent of both systems was consistent and slightly basic at pH of 7.5 for Pisgah and 7.7 for Retrieve which is well within the Jamaican national effluent standards for pH of 6 – 9.

Table 5-10 pH of Water Samples at Pisgah and Retrieve Sanitation Systems

Sample	Arithmetic Mean ($\overline{X}$) mg/l	90% Confidence Interval mg/l	Sample Size (n)
Pisgah First Septic Tank	7.3	$\overline{X} \pm 0.4$	5
Pisgah Wetland Influent	7.4	$\overline{X} \pm 0.5$	5
Pisgah Wetland Effluent	7.7	$\overline{X} \pm 0.1$	4
Retrieve First Septic Tank	7.2	$\overline{X} \pm 0.3$	6
Retrieve Wetland Influent	7.2	$\overline{X} \pm 0.3$	6
Retrieve Wetland Effluent	7.5	$\overline{X} \pm 0.1$	7

Chapter 6 – Cost of Pisgah and Retrieve Sanitation Systems

Capital Costs

Capital costs in U. S. Dollars for the construction of the Pisgah and Retrieve Sanitation systems is presented in Table 6-1. The Pisgah system cost US$ 13,396 and was designed for a flow rate of approximately 2,580 liters/day. Payments were made in 2002 and 2003 at an average exchange rate of J$52 = US$1 (Onada, 2005). The Retrieve system cost US$ 8,699 and was designed for a flow rate of approximately 1,290 liters/day. Payments for the Retrieve system were made in 2002 at an exchange rate of J$49 = US$1 (Onada, 2005).

Table 6-1 Construction Costs for Pisgah and Retrieve Sanitation Systems.

Item	Pisgah (US Dollars)	Retrieve (US Dollars)
Building materials (steel, wood, concrete, pipe)	4,472	3,570
Rock media	2,308	2,449
Septic and water storage tanks	1,686	1,354
Plastic liner	1,256	n/a
Materials Subtotal	9,723	7,373
Jack-hammer (compressor)	2,058	204
Skilled labor	856	306
Community labor	750	816
Labor Subtotal	3,673	1,327
Total	$13,396	$8,699

The Pisgah system utilized a rainwater harvesting scheme of guttering, water storage and gravity delivery. The Retrieve system had a solar powered lift pump to deliver rainwater to an elevated tank when the municipal supply was not in service. The US$4,821 for the solar powered pumping system was not included in Table 6-1. Community labor was utilized at both sites to different degrees. This prevented direct comparison of labor costs between the projects and prevented direct transfer of per unit cost for estimating the expense of future projects. However, the available figures may be valuable for general knowledge and perspective.

The per capital water usage of 1.3 L/p-d at Pisgah versus the 48.3 L/p-d water usage at Retrieve illustrates the difficulty in predicting the appropriate design flow rate for new sanitation systems. Recall that both of these schools had pit latrines before conversion to the flush toilet systems. Both sanitation systems were sized based on reported school populations and the same estimate for per capita water use. It should also be noted that the Retrieve system was designed for approximately twice the number of students than were attending during the evaluation.

Because of the variation in per capita water use, dollar per liter of wastewater treated is not a useful cost metric. A more helpful capital cost comparison is dollar per liter of design capacity. Dividing the Pisgah sanitation system construction cost of US$ 13,396 by the 2,580 liters/day design capacity gives US$ 5.19 per liter per day. Dividing the Retrieve sanitation system construction cost of US$ 8,699 by the 1,290-liters/day design capacity gives US$ 6.74 per liter per day. These costs may be valuable for projecting the capital cost of future designs.

Operating Costs

The operation cost is expected to be low for any sanitation system such as that at Pisgah which uses rainwater harvesting for water supply and gravity instead of electricity for all water flow. The Retrieve system solar panels will not require utility payments although upkeep of an electric pump does increase future costs.

The septic tanks will have to be pumped out by a vacuum truck at some time. It was observed that the solids accumulation in the all of the septic tanks was very low. The depth of sludge and grit in the first chamber of septic tanks at Pisgah was 6 inches on the boys' side and 2 inches on the girls' side. The buildup was in the shape of a pile mostly under the inlet tee. The depth of sludge diminished with distance from the tee. At the time of measurement these two tanks had been serving 205 students (assume 102.5 students each) for two full school years. The depth of sludge/grit in the first chamber at Retrieve was 4 inches under the inlet tee and diminishing with distance away from the tee. This system had been serving 69 students for approximately two and a half school years. These septic tanks had approximately 44 inches of water depth. If this accumulation represented only septic (anaerobic) operation, extrapolation of the sludge/grit depth would indicate that sludge removal with a septic tank vacuum truck might not be necessary for many years. However, the systems operated as aerobic digesters before the aeration equipment was removed just prior to this study, so a sludge accumulation rate under current conditions cannot be accurately derived from the data. The introduction of soaps, oils and particles common in gray water from kitchens and lavatories would increase the rate of sludge buildup.

Subsurface flow wetlands require periodic observation to assure that there is not surface flooding, leaking or other failure. Wetland plants do not need to be harvested. If a harvesting program is employed, the labor costs would be minimal for these small systems.

Chapter 7 – Conclusions and Recommendations

The Pisgah and Retrieve sanitation systems were evaluated to document wastewater treatment effectiveness and the impact of design parameters in local conditions. According to the results from five sets of water quality tests, it was determined that BOD was reduced by 78% at Pisgah and 50% at Retrieve; TSS was reduced by 85% at Pisgah, but increased at Retrieve; total nitrogen was reduced by 95% at Pisgah and 68% at Retrieve; ammonia was reduced by 99% at Pisgah and 97% at Retrieve; total phosphorus was reduced by 97% at Pisgah and 64% at Retrieve; and fecal coliform was reduced by 99.99% at Pisgah and Retrieve. Average effluent nitrate levels remained below 1.0 mg/l for both systems.

Both of the sanitation systems performed to the magnitude expected with a few exceptions. Total suspended solids were generated rather than reduced in the Retrieve wetland, ammonia removal in both of the wetlands was higher than what literature reports for other subsurface wetland and coliform reduction in the Retrieve septic tanks was higher than what literature reports for septic tanks. The effluent sample collection method at Retrieve was believed to be the primary cause for the unexpected TSS results. A normal SSF wetland should reduce TSS at least as well as it reduces BOD (USEPA, 2000). An erroneously high concentration of solids in the Retrieve wetland effluent samples could also have negatively affected the other Retrieve effluent water quality test results. Absorption by wild cane and possibly adsorption by the rock media contributed to the high ammonia removal in both wetlands. Over time the plants and media will age and the concentration of ammonia in the effluent may increase. The high reduction of coliform measured for the Retrieve septic tanks may have been caused by the continuous flow of chlorinated water through the urinal.

Although determining the appropriate design size of a large wetland requires estimating incoming loads and performing calculations using accepted mathematical models, persons designing small subsurface flow wetlands for onsite wastewater treatment in

Jamaica may refer to the hydraulic data presented in this report. The Pisgah wetland evaporated 0.27 USgal/ft^2/d of wastewater and the Retrieve wetland effectively treated a monthly average of 2.6 USgal/ft^2-d (105 L/m^2-d). The 6.5:1 length to width ratio of the Retrieve wetland was found to be too long and narrow to prevent surface flooding when the weekly hydraulic load reached 3.85 USgal/ft^2-d. It would be advisable to size small wetlands for a hydraulic load below 2 USgal/ft^2-d and keep aspect ratios closer to 1:1. The construction of two wetland beds at Pisgah and three beds at Retrieve enhanced the flexibility of these systems by allowing for future growth or bed alternation.

Best practices were followed in designing the septic tanks and wetlands for effective wastewater treatment. The R2RW septic tank design incorporated two chambers, a large surface area in the first chamber, sanitary tees and sufficient total volume for 24-hour liquid retention time. Structural integrity of the soils above the plastic tanks can be enhanced by covering with eight inches of a weak mixture of cement and marl (Cotterel, 2005). Marl in Jamaica is a mixture of limestone with a small percentage of other inorganics and is commonly used in the form of stones with powder. The addition of clean-out risers at both ends of the wetland inlet distributor pipes will facilitate removing any material that may buildup in the pipe over time. Manhole covers should be light enough to allow access, heavy enough to deter inquisitive youth and sealed well to prohibit mosquito breeding. Each wetland bed at Retrieve has a PVC elbow snug fit but not glued onto the outlet drain pipe so the elbow can be turned to change the overflow elevation in the bed. This is a beneficial level control feature. During a recent wetland construction project it was discovered that care should be taken to confirm that there is adequate elbow rotation space.

The labor and materials costs for building the sanitation systems were divided by the design sizes to yield unit costs that may be useful for roughly estimating the cost of future projects. Future labor costs may be significantly higher because community labor was used for portions of these projects. The capital cost for the Pisgah system was US$5.19 per liter of water treated per day and the capital cost for the Retrieve system was US$6.74

per liter of water treated per day. Operation and maintenance costs are expected to be low because these natural wastewater treatment systems require little or no power input. The rate of sludge buildup in the septic tanks was 1 to 3 inches per year, but this may not represent typical septic tank operation since the tanks were previously aerobic. Only black water entered these systems, and the introduction of gray water may affect the treatment performance and rate of sludge buildup.

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Chapter 9 – Appendices

Appendix 1 - Water Quality Analysis Raw Data

1-Sep-2004 **BACKGROUND DATA**

Sample	BOD	TSS	Tot. N	NO3-N	NH3-N	Tot. P	Tot. Coli.	Fecal Coli.	pH
	Mg/l	mg/l	mg/l	mg/l	mg/l	mg/l	MPN	MPN	
PS1	9	41	23.4	0.1	19.8	4.6	2,800,000	2,200,000	6.6
PS2	9	13	15.9	0.9	27.6	4.8	160,000	160,000	7.3
PS2*	6	15	29.8	0.9	28.3	5.3	26,000	21,000	7.3
PS3	5	1	0.7	0.0	0.6	0.4	24,000	1,600	7.4
RT1	9	55	10.4	0.0	7.2	2.2	3,500	2,400	6.6
RT2	6	11	11.8	0.0	7.1	1.8	170	130	7.2
RT3	27	20	8.6	0.2	2.1	1.4	33	33	6.7

* denotes duplicate sample

21-Sep-2004 **SET ONE**

Sample	BOD	TSS	Tot. N	NO3-N	NH3-N	Tot. P	Tot. Coli.	Fecal Coli.	pH
	Mg/l	mg/l	mg/l	mg/l	mg/l	mg/l	MPN	MPN	
PS1	23	162	8.9	0.2	13.6	3.8	2,400,000	n/a	6.9
PS2	20	160	9.6	0.3	14.7	2.5	2,400,000	n/a	6.9
PS3	17	19	0.6	0.6	1.0	0.6	35,000	n/a	7.7
RT1	8	23	1.8	0.3	6.4	1.9	170,000	n/a	7.0
RT2	32	373	4.2	0.1	10.5	2.6	160,000	n/a	7.0
RT3	32	268	1.5	1.0	0.4	0.9	1,700	n/a	7.8
RT3*	36	157	2.8	0.2	0.0	0.7	2,600	n/a	7.8

* duplicate

5-Oct-2004 **SET TWO – NWC Lab**

Sample	BOD	TSS	Tot. N	NO3-N	NH3-N	Tot. P	Tot. Coli.	Fecal Coli.	pH
	Mg/l	mg/l	mg/l	mg/l	mg/l	mg/l	MPN	MPN	
PS1	56	104	24.7	0.0	25.7	5.4	3,100,000	3,100,000	6.9
PS2	26	58	17.7	0.2	18.2	3.9	70,000	70,000	6.7
PS3	11	3	1.5	1.0	0.0	0.3	280	94	7.7
RT1	69	73	23.2	0.0	29.6	5.7	11,000,000	11,000,000	6.8
RT2	16	6	7.8	1.0	6.6	1.8	170,000	170,000	6.7
RT3	20	54	18.8	6.7	1.7	0.7	4,000	2,000	7.4

5-Oct-2004 **SET TWO – NEPA Lab**

Sample	BOD	TSS	Tot. N	NO3-N	NH3-N	PO4-P	Tot Coli.	Fecal Coli.	pH
Method	**5210B**	**2540B**	**n/a**	**4500E**	**n/a**	**4500PE**	**n/a**	**9221E**	
	Mg/l	**Mg/l**	**mg/l**	**mg/l**	**mg/l**	**mg/l**	**MPN**	**MPN**	
PS1	-	58	-	0.04	-	10.5	-	1600	7.0
PS1*	-	64	-	0.77	-	10.2	-	≥ 1600	7.2
PS2	-	96	-	0.08	-	5.4	-	≥ 1600	7.0
PS3	-	20	-	0.04	-	4.8	-	13	7.0
RT1	-	42	-	1.33	-	13.0	-	≥ 1600	6.8
RT2	-	10	-	0.13	-	6.0	-	≥ 1600	7.0
RT3	-	26	-	1.69	-	9.9	-	140	6.9

* duplicate

2-Nov-2004 **SET THREE**

Sample	BOD	TSS	Tot. N	NO3-N	NH3-N	Tot. P	Tot. Coli.	Fecal Coli.	pH
	Mg/l	**mg/l**	**mg/l**	**mg/l**	**mg/l**	**mg/l**	**MPN**	**MPN**	
PS1	68	75	41.3	0.3	115.0	14.2	920,000	920,000	7.7
PS2	23	16	40.5	0.9	72.0	9.7	240,000	240,000	7.9
PS3	12	19	1.5	0.3	0.3	0.2	1,700	1,700	7.6
RT1	27	25	2.3	0.6	3.4	1.8	2,200,000	2,200,000	7.5
RT1*	12	27	9.5	0.5	3.2	1.3	540,000	540,000	7.6
RT2	6	13	2.8	0.5	4.9	1.4	7,000	7,000	7.5
RT3	12	12	16.8	0.2	0.5	1.4	170	170	7.5

* duplicate

16-Nov-2004 **SET FOUR**

Sample	BOD	TSS	Tot. N	NO3-N	NH3-N	Tot. P	Tot. Coli.	Fecal Coli.	pH
	Mg/l	**mg/l**	**mg/l**	**mg/l**	**mg/l**	**mg/l**	**MPN**	**MPN**	
PS1	72	83	30.9	0.2	114.0	20.7	7,500,000	7,500,000	7.7
PS2	n/a	22	30.1	0.6	73.0	12.1	3,500,000	3,500,000	7.7
PS3	n/a	11	2.6	1.0	0.2	0.3	27	17	7.7
RT1	51	24	40.9	0.7	4.0	3.3	940,000	940,000	7.1
RT2	9	6	7.3	0.3	3.5	1.0	130,000	130,000	7.5
RT2*	n/a	3	7.1	0.7	3.5	0.8	130,000	130,000	7.4
RT3	14	12	2.8	1.0	0.0	0.5	34	13	7.4

* duplicate

7-Dec-2004 **SET FIVE**

Sample	BOD	TSS	Tot. N	NO3-N	NH3-N	Tot. P	Tot. Coli.	Fecal Coli.	pH
	Mg/l	mg/l	mg/l	mg/l	mg/l	mg/l	MPN	MPN	
PS1	72	38	60.7	0.1	112.0	20.1	7,000,000	4,900,000	7.5
PS2	38	30	51.0	0.5	110.0	19.6	540,000	350,000	7.9
PS3	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
RT1	75	74	42.2	0.0	31.5	5.6	35,000,000	35,000,000	7.3
RT2	13	4	11.4	0.0	6.0	1.2	220,000	220,000	7.2
RT3	17	105	4.6	0.5	0.0	2.0	36	27	7.4
RT3*	11	80	7.6	0.3	0.0	2.0	350	17	7.4

* duplicate

Appendix 2 - Outlier Data Rejection

	RT3 NO3 (mg/l)	RT2 TSS (mg/l)
Sep 21	1	373
Sep 21	0.2	-
Oct 5	6.7	6
Nov 2	0.2	13
Nov 16	1	6
Nov 16	-	3
Dec 7	0.5	4
Dec 7	0.3	-
X high	6.7	373
X bar (mean)	1.4142857	67.5
X low	0.2	3
std dev	2.356147	149.7047
T = (Xhigh -Xbar)/std dev	2.2433721	2.040684
T = (Xbar-Xlow)/std dev	0.5153693	0.430848
	for n = 7	for n = 6
T values from Standard Methods Table 1010: I (APHA: 1998)	T = 2.10	T = 1.94

The October 5 data for Retrieve wetland effluent nitrate (RT3 NO3) and September 21 data for Retrieve wetland influent TSS (RT2 TSS) were rejected as outliers because the T value of the data exceeded 1% of normal sample discordancy T values according to Table 1010: I in Standard Methods text.

Appendix 3 - Determination of 90% Confidence Intervals for Water Quality Data

Means
Arithmetic means for BOD, TSS, Total N, NO3-N, NH3-N, Total P and pH.
Geometric means for Total Coliform and Fecal Coliform.

Sample	BOD	TSS	Tot. N	NO3-N	NH3-N	Tot. P	Tot. Coli.	Fecal Coli.	pH
	mg/l	mg/l	mg/l	mg/l	mg/l	mg/l	MPN	MPN	
PS1	58	92	33.3	0.2	76.1	12.8	3,498,060	3,199,644	7.3
PS2	27	57	29.8	0.5	57.6	9.6	422,127	378,758	7.4
PS3	13	13	1.6	0.7	0.4	0.3	234	140	7.7
RT1	40	41	20.0	0.4	13.0	3.3	2,044,760	3,362,645	7.2
RT2	15	6	6.8	0.4	5.8	1.5	94,405	84,952	7.2
RT3	20	98	**7.8**	0.5	0.4	1.2	386	73	7.5

Standard Deviations

Sample	BOD	TSS	Tot. N	NO3-N	NH3-N	Tot. P	Tot. Coli.	Fecal Coli.	pH
	mg/l	mg/l	mg/l	mg/l	mg/l	mg/l	MPN	MPN	
PS1	20.741	45.632	19.299	0.114	51.684	7.959	2,912,779	2,787,609	0.41
PS2	7.890	59.676	16.722	0.274	40.568	6.877	1,520,822	1,644,040	0.58
PS3	3.215	7.659	0.819	0.340	0.435	0.175	17,181	950	0.05
RT1	28.856	25.211	18.410	0.302	13.642	1.962	13,695,793	14,649,187	0.31
RT2	10.134	3.912	3.008	0.378	2.614	0.653	71,415	78,771	0.32
RT3	9.966	90.812	**7.094**	0.378	0.624	0.637	1,555	872	0.19

Statistical t Table for Determining Confidence Interval

df \ p	0.4000	0.2500	0.1000	0.0500	0.0250	0.0100	0.0050	0.0005
1	0.3249	1.0000	3.0777	6.3138	12.7062	31.8205	63.6567	636.6192
2	0.2887	0.8165	1.8856	2.9200	4.3027	6.9646	9.9248	31.5991
3	0.2767	0.7649	1.6377	2.3534	3.1825	4.5407	5.8409	12.9240
4	0.2707	0.7407	1.5332	2.1318	2.7765	3.7470	4.6041	8.6103
5	0.2672	0.7267	1.4759	2.0150	2.5706	3.3649	4.0321	6.8688
6	0.2648	0.7176	1.4398	**1.9432**	2.4469	3.1427	3.7074	5.9588
7	0.2632	0.7111	1.4149	1.8946	2.3646	2.9980	3.4995	5.4079
8	0.2619	0.7064	1.3968	1.8595	2.3060	2.8965	3.3554	5.0413
9	0.2610	0.7027	1.3830	1.8331	2.2622	2.8214	3.2498	4.7809
10	0.2602	0.6998	1.3722	1.8125	2.2281	2.7638	3.1693	4.5869

Confidence Interval Sample Calculation:

Determine 90% confidence interval for RT3 Total Nitrogen

$\overline{X} \pm (Z_{\alpha/2})\frac{\sigma}{\sqrt{n}}$ For 90% α = 1-0.9 = 0.1

Using the t-Table

α /2 = 0.05 (this is called p for the table so p = 0.05)

degrees of freedom (df) = sample size – 1, or n – 1 df = 7-1 = 6

$= 7.8 \pm (1.9432)\ \frac{7.094}{\sqrt{7}}\ =\ 7.8\ \pm\ 5.2$

There is 90% confidence that the mean is between 2.6 and 13.0.

Appendix 4 - Conductance of Water Corrected for Temperature

	Conductance (μmho)	
Sample	**17-Aug**	**21-Sep**
Pisgah septic inlet	516	230
Pisgah septic outlet	472	232
Pisgah mid-wetland	438	447
Retrieve septic inlet	245	177
Retrieve septic outlet	352	272
Retrieve wetland effluent	575	387
Retrieve wetland effluent duplicate		392

Appendix 5 - BOD Sample Calculation

Thiosulfate titrant volume is multiplied by 2 because the titrated sample is 100ml instead of standard method 200ml.
Titration of sample on day one: 3.6ml thiosulfate x 2 = 7.2 mgO_2/L
Titration at end of five day incubation: 3.0ml x 2 = 6.0 mgO_2/L
7.2 – 6.0 = 1.2 mgO_2/L depletion
N = 300/(sample size in ml)
For a dilution using a 10 ml sample: N = 300/10 = 30
BOD = N x depletion = 30 x 1.2 = 36

Theoretical BOD Temperature Correction
Based Upon First Order Reaction Kinetics

k_T = reaction rate constant at temperature T
k_{20} = reaction rate constant at 20C, assumed to be 0.23/day
t = time in days
UBOD = Ultimate Biochemical Oxygen Demand
BOD_5 = Five Day Biochemical Oxygen Demand (BOD)

$BOD_5 = UBOD\ (1 - e^{-kt})$
$k_{16} = k_{20}\theta^{T-20}$
Assuming $\theta = 1.056$
$k_{16} = 0.23 \times 1.056^{-4} = 0.185$

If BOD at 16C = 68
$68 = UBOD(1-e^{-0.185x5})$
UBOD = 68 / (1-0.396) = 113
$BOD = UBOD\ (1 - e^{-kt})$
BOD at 20C = $113\ (1 - e^{-0.23x5}) = 77$

Appendix 6 - Total Dissolved Nitrogen Assay Using Potassium Persulfate Digestion

From: D'Ella, C. F.; P. A. Steudler and N. Corwin. 1977. Determination of total nitrogen in aqueous samples using persulfate digestion. Limnol. Oceanog. 22: 760-764.

Preparation of Reagents

Oxidizer: Dissolve 3.35 g Potassium Persulfate and 3.0 g of Sodium Hydroxide in a 500 ml flask. Have approx 250 ml distilled water in the flask before adding dry reagent so the flask does not become too hot.

Buffer: Combine 50.5 ml of 1 Molar Sodium Hydroxide and 15.45 g Boric Acid in a 500 ml flask.

0.3 N HCl: Dilute 24.8 ml concentrated HCl in a 1000 ml flask.

Fixing Samples

In an autoclavable glass vessel with tight screw top, place 20 ml of sample or standard. Add 30 ml Oxidizer and quickly cap. NH4 converts to NH3 and this volatilizes in the high pH. Place samples in autoclave at 100 C for 1 hour. When sample cools, add 3.0 ml of 0.3 N HCl then cap and shake. All precipitate must be dissolved before proceeding. When dissolved, add 4.0 ml buffer and 3.0 ml distilled water. Reagent blanks of distilled water should be prepared as samples each day. Sample is now ready to be analyzed according to the Nitrate Assay protocol. Remember that sample concentrations have been diluted 1:3.

Appendix 7 – Thomas' Most Probable Number Sample Calculation for Coliform

When the number of positive fermentation tubes does not fit into the normal distribution table available in the Standard Methods text (APHA), a calculation using Thomas' formula is used to determine MPN as follows:

$$\text{MPN} = (\#\text{ positive tubes x } 100)\Big/\sqrt{(\text{ml in neg. tubes x total ml})}$$

MPN determination for November 16 sample PS1					
					Thomas'
Range	**# positive**	**Sample**	**ml in neg.**	**total ml**	**formula**
	Tubes	**size (ml)**	**Tubes**		**MPN**
10M	5	0.0001	0	0.0005	
CM	2	0.00001	0.00003	0.00005	
MM	3	0.000001	0.000002	0.000005	
Total	10		0.000032	0.000555	**7,503,753**

$$\text{MPN} = (10 \text{ x } 100)\Big/\sqrt{(0.000032 \text{ x } 0.000555)} = 7{,}503{,}753$$

Appendix 8 - Evapotranspiration Determination

ET = Precipitation + Influent ± Change in Storage – Effluent

Retrieve

Precip. (m^3/wk)	Precip. (gal/ft^2/wk)	Influent* (m^3/wk)	Storage (m^3) S 1	S 2	ΔS	Effluent (m^3)	ET (m^3/d)	ET (USG/ft^2/d)
1.06	0.81	0.61	3.43	2.81	-0.62	0.00	0.33	0.25
1.47	1.12	0.61	2.81	2.81	0.00	0.00	0.30	0.23
						AVG =	0.31	0.24

*Influent volume based on reported daily bucket use

Pisgah

Precip. (m^3/wk)	Precip. (gal/ft^2/wk)	Influent (m^3/wk)	Storage (m^3) S 1	S 2	ΔS	Effluent (m^3)	ET (m^3/d)	ET (USG/ft^2/d)
7.20	2.04	2.93	2.59	4.83	2.24	0.00	0.99	0.27
12.12	3.30	7.93	6.29	8.02	1.72	0.00	2.62	0.71
6.40	1.74	2.38	8.02	5.86	-2.16	0.00	1.56	0.43
7.55	2.06	1.61	5.86	6.12	0.26	0.00	1.27	0.35
3.20	0.87	2.08	6.12	6.12	0.00	0.00	0.75	0.21
5.49	1.50	2.22	6.12	7.24	1.12	0.00	0.94	0.26
0.00	0.00	2.18	7.24	4.57	-2.67	0.00	0.69	0.19
0.00	0.00	1.50	4.57	2.41	-2.16	0.00	0.52	0.14
0.23	0.06	0.88	2.41	0.43	-1.98	0.00	0.44	0.12
0.00	0.00	1.91	0.43	0.43	0.00	0.00	0.27	0.07
						AVG	1.01	0.27

Appendix 9 - Precipitation Data for Proximate Locations