1-1-2009

Nutrient Management in On-Site Wastewater Treatment

Ayanangshu Dey

NUTRIENT MANAGEMENT IN ON-SITE WASTEWATER TREATMENT

By

Ayanangshu Dey

A Dissertation
Submitted to the Faculty of
Mississippi State University
in Partial Fulfillment of the Requirements
for the Degree of Doctor of Philosophy
in Civil and Environmental Engineering
in the Department of Civil and Environmental Engineering

Mississippi State, Mississippi

December 2009

NUTRIENT MANAGEMENT IN ON-SITE WASTEWATER TREATMENT

By

Ayanangshu Dey

Approved:

Dennis D. Truax,
Professor of Civil and
Environmental Engineering
(Director of Thesis)

Benjamin S. Magbanua Jr.,
Associate Professor of Civil and
Environmental Engineering
(Committee Member)

James L. Martin,
Professor of Civil and
Environmental Engineering
(Committee Member)

William H. McAnally,
Associate Professor of Civil and
Environmental Engineering
(Committee Member)

Kari Babski-Reeves,
Assistant Professor of Industrial and
Systems Engineering
(Committee Member)

James L. Martin,
Professor of Civil and
Environmental Engineering
Graduate Program Coordinator

Sarah A. Rajala
Dean of the College of Engineering

Name: Ayanangshu Dey

Date of Degree: December 11, 2009

Institution: Mississippi State University

Major Field: Civil and Environmental Engineering

Major Professor: Dr. Dennis D. Truax

Title of Study: NUTRIENT MANAGEMENT IN ON-SITE WASTEWATER TREATMENT

Pages in Study: 205

Candidate for Degree of Doctor of Philosophy

Groundwater and surface water contamination has been linked in the past to inadequate or failing on-site wastewater treatment and disposal systems. The on-site wastewater systems installed in coastal areas have more potential for inflicting this kind of environmental damage. This work studied the regulatory compliance and environmental protection of the four types of on-site wastewater disposal systems found on the Mississippi Gulf Coast; i.e., vegetative rock filter, subsurface drip irrigation, sand mound, and sprinkler systems, by statistical techniques. Compliance was also evaluated for groundwater samples collected from monitoring wells installed at four corners of a disposal field. This work eventually culminated in formulation of strategy for modifying the aerobic treatment prior to disposal to help reduce nitrogen loading on the discharging environment. Process modeling and simulations were performed to optimize conditions for biological nitrogen reduction in the treatment unit by efficient management of aeration. Two separate proposals were developed, such as either running the aerator unit in a low operating dissolved oxygen concentration or intermittent aeration mode.

ACKNOWLEDGEMENTS

I would like to thank Dr. Dennis Truax, my major advisor, for his continuous guidance, supervision, and support in accomplishing the research effort and drafting this dissertation. I am thankful to my other committee members, Dr. Benjamin Magbanua Jr., Dr. James Martin, Dr. William McAnally, and Dr. Kari Babski-Reeves for their critical review, help, and advice. I sincerely acknowledge the help of Mr. Joe Ivy as and when needed during the entire course of my program. I also thank Kris, Lannie, David, Morgan, and Taylor for their assistance.

At a personal level, I most gratefully acknowledge the profound influence and encouragement of a few individuals whom I will always remain indebted to; my high-school teachers Mr. Timir Prakash Deb and Late Mr. Rathindra Nath Mazumdar, my big brother Mr. Gautam Sinha, and aunt Ms. Rekha Chowdhury. I also thank all my friends and colleagues here and back home.

Lastly, nothing like this would have been possible without the patience and understanding of my parents, Mrs. Nandita Dey and Mr. Amar Nath Dey, to whom I dedicate this endeavor.

TABLE OF CONTENTS

APPENDIX

LIST OF TABLES

LIST OF FIGURES

LIST OF ABBREVIATIONS

AF	Anoxic Time Fraction
ANOVA	Analysis of Variance
APHA	American Public Health Association
ASM1	Activated Sludge Model no 1
ASP	Activated Sludge Process
ASTM	American Society for Testing and Materials
ATU	Aerobic Treatment Unit
b_A	Decay coefficient for autotrophic biomass
b_H	Decay coefficient for heterotrophic biomass
BNR	Biological Nitrogen Removal
BOD	Biochemical Oxygen Demand, 5 day, at 20 ^{0} C
C_{95}	95% of the values falling either at or below this concentration
cfu	colony forming unit
COD	Chemical Oxygen Demand
CSTR	Completely Mixed Stirred Tank Reactor
CT	Cycle Time
C_u	Uniformity coefficient
d_{10}	Grain size that is 10% finer by weight (effective size)
d_{60}	Grain size that is 60% finer by weight

DO	Dissolved Oxygen
FC	Fecal Coliform
f'_D	Fraction of biomass leading to debris
FDA	Food and Drug Administration
HRT	Hydraulic Retention Time
IA	Intermittent Aeration
$i_{N/XB}$	Mass of nitrogen per mass of COD in biomass
$i_{N/XD}$	Mass of nitrogen per mass of COD in products from biomass
K	Hydraulic conductivity
k_h	Maximum specific hydrolysis rate
K_{NH}	Ammonia half-saturation coefficient for autotrophic biomass
K_{NO}	Nitrate half-saturation coefficient for denitrifying heterotrophic biomass
$K_{O,A}$	Oxygen half-saturation coefficient for autotrophic biomass
$K_{O,H}$	Oxygen half-saturation coefficient for heterotrophic biomass
K_S	Half-saturation coefficient for heterotrophic biomass
K_X	Half-saturation coefficient for hydrolysis of slowly biodegradable substrate
k_a	Ammonification rate
LL	Liquid Limit
μ_A	Maximum specific growth rate for autotrophic biomass
μ_H	Maximum specific growth rate for heterotrophic biomass
MDEQ	Mississippi Department of Environmental Quality
MDH	Mississippi Department of Health
η_g	Correction factor for μ_H under anoxic conditions
η_h	Correction factor for hydrolysis under anoxic conditions

NPS	Non-Point Source
ρ	Porosity
PDF	Probability Density Function
PL	Plastic Limit
PVC	Poly-Vinyl Chloride
QAPP	Quality Assurance Plan and Procedure
QA/QC	Quality Assurance/Quality Control
R	Sludge Recycle Ratio
RO	Oxygen Requirement
SM	Standard Methods
SND	Simultaneous Nitrification Denitrification
S_{NH}	Free and Unionized Ammonia Nitrogen
S_{NO}	Nitrate and Nitrite Nitrogen
SRT	Solids Retention Time
S_S	Soluble Chemical Oxygen Demand
Std. Dev.	Standard Deviation
S_{TN}	Soluble Total Nitrogen
θ	Hydraulic Retention Time
θ_X	Solids Retention Time
TKN	Total Kjeldahl Nitrogen
US EPA	United States Environmental Protection Agency
USDA	United States Department of Agriculture
Y_A	Yield factor for autotrophic biomass
Y_H	Yield factor for heterotrophic biomass

CHAPTER I

INTRODUCTION

On-site Wastewater Management (also termed as Decentralized Wastewater Management) can be defined as the collection, treatment, and reuse of wastewater coming from individual homes, cluster of homes, subdivisions, and isolated commercial facilities at or close to the point of wastewater generation (Tchobanoglous et al., 2004). The objectives of using such type of treatment and disposal techniques may be identified as, safeguarding public health from potential pollution by the contaminants of wastewater, protecting the receiving environment (e.g. groundwater) by ensuring that adequate treatment is received by the effluent in this process before disposal, and reduction in cost of treatment by segregating it from any centralized treatment system by retaining the wastewater and solids at their point of origin either through reuse or adoption of a better disposal technique (Tchobanoglous, 1998).

Decentralized wastewater treatment forms an integral part of the present arena of wastewater management. These types of treatment typically serve separate or cluster of individual homes, large capacity septic systems, and small collection and treatment system such as package plant. Individual or clustered homes are typically located in the fringe areas of a city. With this increase in distance, it also becomes gradually difficult, complicated, and sometimes impracticable to integrate and connect such sites or plots with the centralized sewerage system. Several factors can be identified for this; for example, large distance of housing units from collection system, difficulty in ensuring gravity conveyance of sewer to any vantage collection point due to existing topography, long haulage of sewage making it turn septic, requirement of additional number of sewage pumping stations for conveyance. Apart from the above there may be other reasons also; e.g. less flow in initial stretches, non-development of self-cleansing velocity requiring frequent maintenance, overloading of the downstream facilities if not designed to cater for this added quantity of sewage, etc.

Under such circumstances, the de-centralized wastewater system might prove to be the only available alternative for treatment and disposal of wastewater from such individual, and segregated housing units to comply with applicable discharge standards. It can work out to be a cost effective alternative as well, as opposed to incurring substantial capital cost in laying long stretches of sewers, building pumping stations, installing pressure mains, integrating such facility with the existing system, etc. The treatment and disposal techniques applied in decentralized wastewater treatment are not very sophisticated systems and are relatively cheap in terms of installation cost and maintenance thereafter.

Generally speaking, a single decentralized unit comprises three separate components for collection, treatment, and disposal.

- Collection – The collection component is responsible for collecting and conveying the wastewater generated from source to the treatment unit. It can be either gravity conveyance or pressure transmission (i.e. by pumping) depending upon requirement and natural topography of the site.
- Treatment – Mainly two treatment options are available for wastewater, aerobic and anaerobic. Septic tanks are used in most of these installations employing anaerobic biodegradation to stabilize the organic contents. Aerobic treatment units

(ATU) are sometimes installed to render aerobic biological treatment to the influent. Following this, these tanks are used to settle out the sludge formed and other settleable solids from the influent, make any oil and grease float to the surface for removal, and store the partially treated wastewater prior to its disposal.

- Disposal – After treatment, wastewater is disposed over a defined area of land to undergo further polishing by the soil and vegetative cover as it gradually percolates through soil before reaching groundwater. Common disposal practices applied are subsurface drip irrigation, sprinkler system, mound system, constructed wetland (or rock bed filters), and other forms of land treatment. Sometimes disinfection (by chlorination) is applied to the partially treated effluent before its application to disposal field.

The disposal techniques employed in de-centralized systems are mainly dependent on the natural attenuation capacity of the existing soil matrix, vegetative cover, and microbial activity in the disposal site to deplete the contaminant level. In this respect, a number of land treatment techniques have been proven to be effective. Some decentralized systems can be designed to discharge the effluent into a receiving surface water source after treatment. However, there is little or no provision kept for regulating this discharge from the standpoint of potential contamination of environment. Hence, it is required to evaluate these techniques for their adequacy in meeting the current discharge standards, if any. Finding suitable modification(s) of the system for reducing nutrient load on the receiving water body is also needed.

In the past, several proposals for nitrogen removal from on-site wastewater management system were put forward. Such nitrogen removal systems proposed under physical or chemical treatment were ion-exchange or reverse osmosis; and several other biological nitrogen removal systems proposals put forward were, extended aeration, aerobic or anaerobic trickling filter, peat filter, re-circulating sand filter, re-circulating sand filter with anaerobic filter, etc. (Whitmyer et al., 1991).

Septic tanks are not suitable for all on-site wastewater system application and used for replacing failing septic systems, which are a major source of groundwater pollution in some areas (US EPA, 2000). Linsley (1992) concluded that approximately two-thirds of all land area in the United States is estimated to be unfit for septic system installation. In view of the above and several documented advantages of aerobic treatment units over septic tanks (US EPA, 2000), current research effort has been focused on working out modifications in the aerobic units to supplement nitrogen removal.

Currently, the aspect of nitrogen reduction is left entirely to the disposal fields and it is expected that some percentage of nitrogen removal can actually be achieved in the disposal fields. In this research, a strategy has been proposed to share the burden of nitrogen reduction between the aerobic treatment unit (ATU) and the disposal field to further enhance nitrogen reduction from wastewater. This has been supported with predictions by process modeling and simulation of biological nitrogen removal to work out two possible options of better aeration control philosophies to complement nitrogen removal in on-site wastewater management systems.

CHAPTER II

BACKGROUND AND STUDY OBJECTIVES

This chapter briefly delineates the background, present status, extent of application, and research initiatives of on-site wastewater systems, first from overall US perspective which then leads to its status in the state of Mississippi. The objectives of the study have also been identified to conclude the chapter.

Brief History and Present Perspective

Treatment of wastewater in the United States has been a catch-up phenomenon employed only when required by existing legislation to protect the public health. In the 1970s and 1980s, large federal investments in the construction of wastewater facilities focused primarily on large, centralized collection and treatment systems. This effort did not recognize the benefits of properly managed decentralized wastewater systems in achieving the goals of the Safe Drinking Water Act (1974) and Clean Water Act (1977).

Development and Decline

The use of decentralized wastewater systems, such as septic tanks, for primary treatment of wastewater began in the late 1800s, and the discharge of septic tank effluent into gravel-lined subsurface drains became a common practice during the middle of 20th century. During the 19th century, public concern for the effects of raw wastewater discharge on health and well-being of expanding population increased, and communities began to plan for and construct sewage collection and treatment systems. Public health departments were subsequently entrusted with to promulgate the first on-site wastewater "disposal" laws then mostly based on soil percolation rates. During the 1950s, states started adopting laws for upgrading design and installation practices of on-site system (US EPA, 2005).

Despite such improvements, many state regulations depended on stipulated design standards which required existing site conditions to match the capabilities of the system to be installed. The approach should have been reversed, where the system would be designed and crafted to suit the site conditions ensuring adequate and proper treatment. The underlying philosophy guiding this strategy was based on the fact that the on-site treatment was viewed as a temporary arrangement and centralized system of wastewater treatment would be available in future. This resulted in a variety of codes and regulations by the state and local agencies to control on-site systems (US EPA, 2005).

Though, the 1970s experienced an increase in research and technological development that created several alternative on-site wastewater treatment techniques capable of meeting secondary and advanced level of treatment, the management aspect of these systems (e.g. site evaluation, installation, operation and maintenance), and also the awareness of the property owners and regulators remained at the pre-1700s level of sophistication. This resulted in many systems being sited, devised, installed, operated, and maintained based on obsolete codes, and standards leading to their malfunction (US EPA, 2005). Eventually, on-site systems were reported to be the most common source of fecal bacteria contamination in groundwater throughout most of the nation (Perkins, 1989).

Re-evaluation by EPA

In 1996, Congress directed the EPA to develop a report addressing the key issues, and these were: (a) the potential of on-site system to make more efficient use of the limited funding available for wastewater infrastructure, (b) appropriateness of these systems as an alternative to centralized treatment, and (c) actions to be taken by EPA to implement these alternatives. In its "Response to Congress on Use of Decentralized Wastewater Treatment System", (US EPA, 1997) EPA concluded decentralized systems can protect public health and the environment, have lower capital and operation and maintenance (O&M) cost for rural communities, are appropriate for a variety of site conditions, and are suitable for ecologically sensitive areas if managed adequately.

Based on current information, the facts about decentralized systems are (US EPA, 2005):

- These systems are serving 25% of the US population,
- Representing 10% of the wastewater flow generated in the US (Bradley, 2009),
- These are applied in about one-third of all new housing and commercial development, and
- Such techniques are typically utilized in rural areas, however, more than half of the 25 million systems are found in suburban areas.

Decentralized Systems in Mississippi

The state of Mississippi is not an exception to the fact and according to 1990 Census, 42% of individual residences in Mississippi have no access to centralized public sewage disposal systems and rely on individual onsite disposal systems. Approximately 85% of these systems are of conventional septic tank and soil absorption field type (MDH, 2002).

As previously mentioned, this prevailing reliance on onsite wastewater treatment was initially based on the assumption that centralized wastewater collection and treatment services would be available in the future. However, with increase in research and technological advancement, a number of alternative decentralized systems capable of meeting secondary and advanced levels of treatment were developed. These systems if properly sited, designed, installed, and maintained could not only protect public health and environment, it is also a cost-effective option for low density communities and could be adapted to suit different sites and field conditions.

On the other hand, if decentralized systems are poorly designed, operated, and managed this can cause significant and widespread nutrient and microbial contamination to ground water (US EPA, 1998). Failed sewage effluent drain field systems become a health hazard when the effluent breaks through the surface of the ground, or contaminates groundwater or surface water. It is important to note that about 12% of housing units in Mississippi (in 1990) relied on private wells for drinking water (MDH, 2002). Because of their proximity to onsite disposal systems, these wells are more likely to be exposed to contamination than community water systems. Failed on-site systems also contribute to non-point source pollution. Surface waters statewide are affected, and particularly the coastal waters of Mississippi. Commercial shellfish harvesting waters are subject to closure when fecal coliform organism reaches certain levels. Many disease outbreaks in the United States traced to drinking untreated groundwater are caused by intrusion of sewage from onsite wastewater disposal systems. It has been estimated that (US EPA, 2005):

- About 10% to 20% of all on-site systems are not adequately treating wastewater (actual failure rates remain unknown).
- Half of the decentralized systems are more than 30 years old and more likely to malfunction.
- Septic tanks are the second greatest threat to ground water quality (as viewed by state water quality agencies).

Future Challenges

After its initiation in the 1950s, presently the states are still saddled with the responsibility of regulating onsite wastewater treatment, this time with more stakeholders such as the Environmental Protection Agency (EPA); National Environmental Health Association; United States Department of Agriculture (USDA) and a host of others. The Mississippi Department of Environmental Quality (MDEQ) is working to obtain the USEPA approval for its Coastal Non-Point Source (NPS) program.

In a much broader sense and from the standpoint of ensuring that adequate treatment is received by the wastewater before reaching the environment, its reliability, sustainability, and potential wastewater reuse, the challenges and opportunities facing on-site wastewater treatment have been identified as (Tchobanoglous et al., 2004):

- Providing protection of public health and environment,
- Overcoming a stigma attached to it due to its past performance and failure of inadequately designed and poorly maintained systems,
- Playing a vital role in water resources management and sustainability (i.e. wastewater reuse),

- Helping to create a paradigm shift from effluent disposal to water reuse, and finally
- Integration into existing centralized system.

Study Objectives

There were two separate aspects of this study, the first being evaluation of disposal techniques, and the second was formulation of strategy to enhance nitrogen removal in on-site treatment. The basic objective of first part covered performance evaluation of existing disposal techniques. The activities conducted for this and analyses of collected data to conclude about their assessment are listed in the next section.

The other important aspect addressed in this study is nutrient management or more specifically the issue of nitrogen removal in on-site wastewater management systems. Two probable control techniques for aerobic treatment units (ATU) were formulated worked out and suggested based on process modeling done for biological nitrogen reduction in order to supplement nitrogen removal in on-site systems. These two control philosophies were aimed at efficient management of aeration in the ATU to:

- Create appropriate environmental conditions under low dissolved oxygen concentration where aerobic nitrification and anoxic denitrification could occur simultaneously, or
- Intermittent operation of aerator to alternatively promote aerobic condition (for autotrophic nitrification) and anoxic condition (for heterotrophic denitrification).

Simulations of these two separate processes were performed by using standard software package for biological process modeling to investigate various aspects of their occurrence and performance as listed below:

- Identification of appropriate operating conditions for effective nitrogen reduction,
- Factors that would most critically affect the occurrence and operation of these processes,
- Finding the ranges of important operating conditions that would inflict significant nitrogen removal,
- Translating these collected information in terms of its use in an on-site wastewater treatment perspective, and
- Formulation of suggested modifications.

Project Activities

In line with the objectives of this research initiative, the project activities undertaken could be categorized under two headings, namely, "evaluation" and "simulation". The funding for the "evaluation" study was made available from Mississippi Department of Health (MDH) and Mississippi Department of Environmental Quality (MDEQ), with the Department of Civil and Environmental Engineering (CEE) of Mississippi State University (MSU) playing a lead role. While the process "simulation" dealt with modeling of biological nitrogen removal process mentioned earlier and further detailed in chapter 4, the "evaluation" part comprised of the following activities:

- Identification of representative sites in the coastal region of Mississippi that adequately represent the diversity of sub-surface conditions and wastewater treatment options that employ leaching fields for disposal of treated wastewater;
- Installation of soil water samplers (lysimeters) and groundwater samplers (monitoring wells) at the designated sites,
- Characterization of the designated sites with regard to sub-surface morphology,
- Collection of samples periodically from the lysimeters and monitoring wells and determination of sub-surface hydrologic conditions corresponding to the sampling activity,

- Performing selected constituent analyses on collected samples,
- Compiling, correlating and analyzing the information collected in the context of an assessment of the effectiveness of current MDH regulations,
- Identifying the extent and dependence of contaminant removal (organics and nitrogen) with respect to variation in weather conditions,
- Looking into the aspect of possible groundwater contamination due to transport of pollutants from disposal fields to groundwater, and
- Investigating if there is any requirement of revising the existing regulations of compliance depth.

CHAPTER III

LITERATURE REVIEW

This chapter will contain the literature review and other relevant information related to the study. It is divided into four separate sections. The first section comprises brief descriptions on four types of disposal techniques that were originally planned to be evaluated in terms of their performance of contaminant removal and compliance to current MDH regulations. These four types of disposal techniques are sub-surface drip irrigation, sprinkler system, sand mound, and rock plant filter. The next section delineates the potential drawbacks of rock plant filter system based on past experience and documented studies. The following section encompasses how the field data will be used to evaluate the performance of these systems and pinpoint the inadequacy in their presumed level of operation. Lastly, strategy for formulating possible system modifications for enhanced nitrogen reduction is described. This section includes review of available literature on the subject of using innovative biological nitrogen removal technique by better control and management of aeration in conventional activated sludge system.

There are certain common issues involved in the design and installation of most disposal techniques. These can be referred to as diverting run-off from the disposal area, maintaining stipulated vegetative cover over the soil surface, fixing up the area required by soil properties and texture, design the system to ensure even effluent distribution as far as possible, restriction on type of land where such fields can be located, periodic inspection of the distributing pipes to avoid clogging problem, specified distance of the field from other property, water line, wells, dwellings, roads, sensitive areas, etc. Important design criteria of these disposal systems based on the regulations issued by the Mississippi Department of Health (MDH), is summarized below for reference.

Subsurface Drip Irrigation

In subsurface drip irrigation system, partially treated wastewater is distributed over a demarcated disposal area by small diameter perforated pipes buried at shallow depth for further treatment. This type of surface disposal system comprises dosing chamber fitted with pumping unit and filter (on the downstream of treatment unit), perforated pipe for distribution of wastewater, disposal field, and other necessary equipment. There are three basic design principles, such as uniform distribution of wastewater, alternative dosing and resting cycles, and shallow placement of dosing trenches (MDH Design Standard, 1997b). A few important aspects of designing subsurface drip disposal system are as follows,

- General requirements – This technique of disposal can be installed in soil types that prohibit implementation of other types of subsurface disposal system. Before designing the system, suitability of the site has to be demonstrated by approved soil testing, characterization, and other topographical features. Where the soil and site conditions are inadequate to support such a disposal system, the system can be placed in a fill. The fill material, if used, should be sandy loam type and incorporated into the existing soil surface by plowing.
- Location – Certain restrictions have been put in locating the irrigation field facility at any piece of land. Adding to this, in case the existing soil has a restrictive horizon (e.g. bedrock, clay, chalk, etc.) within 2 ft of the surface, a min 6 in of unsaturated soil shall be maintained between the ground surface and

groundwater table. In other cases (i.e. no restrictive zone within 2 ft) this distance will be of 12 in.

- Dosing chamber and pumping – The dosing chamber must be designed to have a capacity of 1.5 times the estimated daily flow equipped with an audible high water alarm, and a self-opening relief valve. The dosing chamber will have a grade level access manhole for maintenance and separate vent pipe of min 1 in diameter. It will be made of materials resistant to corrosive effects of wastewater and sealed with suitable sealant to avoid entry of run-off water.

  The pumps for dosing the wastewater will be time based and not demand based. They will be capable of dosing the drip pipes a minimum of 6 or more equally spaced cycles over one day. And each dosing volume will be calculated by dividing the maximum daily flow by the no of cycles employed. The capacity of the pumps, pipes, and other appurtenances will be sized accordingly and will be of suitable materials. A filter has to be provided before distribution to filter the effluent in order to avoid clogging of the distribution pipes.

- Drip field and lines – Apart from the normal consideration for placing the drip disposal field, a min 12 in depth has to be maintained between the bottom of the drip disposal system and seasonal water table, if there is a restrictive zone within 5 ft from the soil surface. If not, then this distance should be 24 in. Drip pipelines should be installed at a depth of 6 in to 8 in from the ground surface with the maximum depth not exceeding 18 in. There shall be a min 12 in of separating distance between the water table and the restrictive layer of soil. Such lines can be installed in trenches dug either by a trenching machine or approved plowing method. Drip lines should not follow contour lines or installed perpendicular to the slope. The min lateral spacing between two pipes will be 2 ft There has to be a system for flushing the distribution lines to prevent clogging by solid built-up inside these lines over time.

Spray Irrigation Disposal

The spray irrigation disposal system is a type of surface disposal technique having a defined area over which wastewater is uniformly sprinkled. The system comprises small diameter pipe (sprinkler laterals) connected to pop up or impact spray irrigation heads, capable of uniformly distributing the effluent at a relatively low rate to avoid soil saturation over the dispersion area (MDH Design Standard, 2006). The two basic features of such technique that distinguish it from some of the others are periodic dosing and resting cycles and uniform distribution of wastewater over disposal field. Other requirements are,

- General requirements – The suitability of the site for placing a spray field must be investigated by conducting acceptable soil permeability test, site conditions, and other topographic features. Care has to be taken in minimizing the chances of human contact with spraying of treated effluent over the spray field. In case fill material is used for spray field to counter soil and site limitations, such fill have to be of a sandy loam texture. Criteria and standards have also been specified for the dosing chamber for retaining the effluent before its distribution and also for pumps used for spraying this over the disposal field.

- Soil and site evaluation – The critical issues involved in evaluation of any particular site for placing the spray field are depth of groundwater table, site slope, restrictive horizon underneath, soil determination up to 5 ft depth or restrictive horizon, soil texture and color (as per NRCS for easy drainage), surface run-off, flooding, and available space meeting all requirements.

- Location – Certain restrictions have been put in locating the spray field facility over any piece of land. Further, in case the existing soil has a restrictive horizon (e.g. bedrock, clay, chalk, etc.) within 2 ft of the surface, a min 6 in of unsaturated soil shall be maintained between the ground surface and groundwater table. In other cases (i.e. no restrictive zone within 2 ft) such a distance has to be 12 in.

- Treatment and disinfection requirements – Wastewater should be treated aerobically before disposal by this system to comply with current standards, avoid any odor problem, and maintain esthetics. It has been stipulated that since in such disposal technique treated wastewater is spread over ground surface, it should be adequately disinfected by relevant standard methods prior to such application.

- Spray field – The size of the spray field has to be determined by soil texture and slope of the site. Maximum precipitation rate should not exceed 0.25 in/hour for any spray field. Even distribution of the effluent is to be ensured by suitable design and sprinkler installation in the field. Spray fields can not be installed in drain ways, swamps, marshes, floodplain, depressed landscape, and other areas.

It would be worthwhile to indicate that the two systems selected in this study did not have any installed disinfection system.

Elevated Sand Mound Disposal

Mounds are sand filters which are dosed with partially treated wastewater under pressure and they discharge directly into the natural soil. Mounds are constructed above the ground surface and their main target is to render adequate treatment to the wastewater percolating down to produce an effluent equivalent to, or better than that from a conventional on-site disposal system, before it reaches groundwater. These systems are adopted for overcoming site restrictions like slow or fast permeable soils, shallow soil cover over restricted bed rock, and high groundwater table (US EPA, 1999). The use of sand mound system is restricted for wastewaters having less than 220 mg/L of BOD or 145 mg/L of TSS (with particles not exceeding size 1/8th in). This type of system consists of a dosing chamber (pressure-distribution component) and sand mound (MDH Design Standard, 1997a). Some important issues of its design and implementation are given here.

- General requirements – The sand mound comprises four different components, namely filter media (sand), an absorption area, a distribution system, and a soil cap and topsoil cover. The treatment received by wastewater in sand mounds is affected by influent strength and characteristics, soil moisture levels, the type of receiving soil, and soil loading rate. The acceptable pre-treatment unit for mound systems will be septic tank with approved filter or aerobic treatment unit.

- Dosing chamber – The dosing chamber should have a storage capacity of min 750 gallons or twice the daily flow (whichever is larger) and it will be fitted with an audible or visual high water alarm. It will be provided with a grade level access manhole for maintenance and a separate vent pipe of specific diameter. The chamber will be made of materials resistant to corrosive effects of wastewater and sealed to avoid entry of run-off water by suitable sealant. The dosing can be either time based or demand based, though time based systems are preferred. One dosing volume should not exceed the maximum daily flow divided by the no of dosing cycles.

- Distribution system – The pressure distribution system of the mound has three components, a pressurized distribution manifold (1 in to 1.5 in diameter) to receive effluent from the pump, field drain pipe to contain the pressure manifold (4 in diameter) with perforations pointing downwards, and distribution media (0.5 to 2.5 in of gravel, 1 foot deep).

- Site requirements – Apart form other aspects of field conditions for which such type of system become applicable, slope limitations are more stringent for mound system than they are for other conventional disposal techniques. Seasonal water table cannot be within 6 in from the ground surface if the soil has a restrictive horizon. If it does not, the water table should not be any closer than 12 in from ground surface. However, for all cases there has to be a separation of 24 in between the bottom of the adsorption area and groundwater table. There are also stipulations given for setback of the area where a mound is to be constructed.

Rock Plant Filters

The idea of using rock filter beds was developed in Kansas in the early 1970s primarily aimed at polishing lagoon effluents of algal cells rather than providing complete treatment of wastewater. There were about 20 operating systems in the US, most of them constructed between 1970 to 1985, having design flows varying form 150 m^3/d to 19,000 m^3/d (US EPA, 2002). This original concept was later modified for adoption as a septic tank effluent disposal technique in on-site wastewater management. Such units then started to be used for individual homes to other small flow sources. It was also termed as "constructed wetlands" being analogous to natural wetlands and targeted at imparting stipulated level of treatment.

In rock plant filter, plants are grown over submerged rock surfaces to remove contaminants through a filtering process. After having developed and researched, this technique received interest as a practical method of wastewater treatment in both municipal and industrial scale, and contaminated site remediation. The system is expected at improving previously treated wastewater by removing suspended solids and nutrients (KSU bulletin, 1998). On emergence of this constructed wetland system and stringent ammonia standards set for wastewater disposal, application of rock filters for polishing of lagoon effluent have reduced over the years (US EPA, 2002). A few rock plant filters in the Midwest date back to the 1990s, but most units in Kansas and surrounding states were installed after 1993. The system is relatively cheap, has no energy cost (except for pumped vertical flow beds), and simple to operate. However, there still exists a dearth of knowledge and operational data to ensure consistent effluent quality that is in compliance with the discharge regulations (KSU bulletin, 1998).

Brief System Description

The most common configuration of rock plant filter is horizontal flow system where the influent enters from one end of rock-bed and treated wastewater exits from other end. In vertical flow system, wastewater is applied from the bottom (normally achieved by pumping) and effluent is collected from the top (US EPA, 2002). Submerged flow systems have nearly been exclusively used for individual homes with most of states' guidelines specifying submerged rock-filter beds. In submerged flow system, water level is maintained at least 2 in below the average rock surface without any requirement for securing the facility either from children or animal (KSU bulletin, 1998).

The different components of the system comprise adequately functioning septic tank, lined treatment bed stacked with rocks and appropriate wetland plants (the rock plant filter), water level controller, unlined absorption bed filled with sand and wetland plants or subsurface absorption system and a seasonal overflow basin, if required by the prevailing soil conditions (KSU bulletin, 1998). The size and type of absorption bed depends mostly on site conditions, and soil properties. Provision of an overflow basin after the absorption field might be required in case flow from extra rainfall in wet seasons or very poor internal drainage capacity of soil is encountered. It is important that adequate water level be maintained in the rock plant filter and absorption cell so that plant growth in the units is healthy enough to impart optimal level of treatment to the wastewater.

- Design Guidelines or Standards – Despite having rock-filter systems for more than 20 years, there is yet any consensus on its design procedure. The operation of rock plant filters in various states has shown variation and adoption of design standard varies from one state to another. Important design aspects and standards as stipulated by the Mississippi DOH are discussed here (MDOH Design Standard, 1997c). Factors that should be considered in design are soil depth and permeability, seasonal variation in groundwater levels, surface topography, lot size and shape, shading by trees, and owner's preference and attitude.

- Hydraulic and Organic Loading - Typical hydraulic loading rates are adopted as 120 to 150 gallons/day/bedroom, however thee rates can be increased or reduced based on actual usage by appropriate regulatory agency. For home system, an organic load of 0.085 lb BOD/day/person can be used assuming 0.17 lb BOD average daily organic loading/day/person and 50% BOD removal in the septic tank.

- Dimensions - Surface area required for the wetland is calculated by adopting a surface hydraulic rate of 1.3 ft^2/gpd for unrestricted area and 0.87 ft^2/gpd for restricted small area. It has been stipulated to use hydraulic rate of 1.3 ft^2/gpd for cold climate. Cross-section area of the bed can be calculated based on hydraulic loading and Darcy's Law. A comparatively low hydraulic gradient up to 1% (assumed equal to the bed slope) and a conservative long-term hydraulic conductivity of 850 ft/day can be used. A hydraulic gradient of 0.5% can be assumed in case of a flat bottom bed. Bed slopes of 2% or higher can be used for sloping lots to optimize cut and fill. The criteria of 1 ft^2/0.05 lb BOD/day can be assumed for finding cross-section area based on organic loading. The larger of the two values for cross-section area is to be adopted. The length, width, and depth of the filter beds can be calculated based on these values obtained and site conditions.

- Substrate (Filter media) - In unrestrictive area, to ensure flow of wastewater through the most effective portion of plant root system, 12 in depth of filter media can be taken. For restrictive small area and colder climatic conditions such depth can be increased to 18 in. A 3 in layer of mulch on the top of the media can be used to ward off potential odors, prevent reflective sun scalding of vegetation, and for visual aesthetics. The most common media used for rock beds is sized and washed gravel. It is preferable to use gravels with rounded surface such as pea gravel and not to use crushed limestone unless it is the only alternative available. Sizes of gravel used for the main substrate is AHD sizes 8 through 9 (average diameter ¼ in and 1/8 in, respectively). Though smaller size is preferred, larger size stones (e.g. AHD sizes 6, 67, or 7 – 1/2 in to 1/8 in) can be used based on local availability. Bigger size stones of 2 in to 4 in can be used around the inlet distributor and the outlet collector pipes to minimize probability of clogging.

- Vegetation - Use of plant species naturally growing in the region and selection in terms of extensive vertical and lateral root growth are preferable. Suggested species include, Typhaceae (cattail family), Cyperacaea (sedge family), Graminear (grass family), and Junacaea (rush family) has been successfully used at several municipal system. There are other species also, and some ornamental plants can be used for visual aesthetics. It has been proposed to plant vegetation during spring to early summer to obtain maximum possible growth before the onset of winter. Adequate spacing of plants, periodic pruning, adjustment of water level, placing mulch to keep the root portion in water, are some of the issues to be taken care of after the system becomes operational.

Potential Drawbacks of Rock Plant Filters

The imminent benefits of installing a rock plant filter system include, system being put in place with relatively less cost when compared to other techniques and simple operation and maintenance. However, its implementation and operation in various states over the years under different soil properties, site conditions, and supported by research initiatives, some inherent drawbacks of the system have been identified as summarized below.

- Inconsistent Effluent Quality – A serious limitation of the rock plant filter system is that they fail to maintain a consistent effluent quality of 30 mg/l BOD and SS to comply with the discharge standards. There is yet no consensus on design procedures for this system (US EPA, 2002). Rock filter performance and design have varied widely over the years based on the data available on its operation and research in various states like Kansas, Illinois, Oregon, Missouri, Louisiana (Middlebrooks, 1988) that have shown spatial and temporal inconsistency over meeting the requirement of 30 mg/l BOD and TSS. Middlebrooks (1995) also concluded that there is insufficient knowledge about mechanisms involved in the filtration process of this system and quantitative impacts of various design parameters on its performance before any reliable estimates can be made about its operation.

- Plugging and Serviceability – The primary removal mechanisms involved in a rock plant filter are believed to be sedimentation, floatation, and interception (O'Brian and McKinney, 1979). Suspended solids contained in the septic tank effluent undergo microbial degradation which can be either anaerobic or aerobic depending on the seasonal variation and nature of micro-environment prevailing on water surface of the rock filter. The refractory materials are accumulated within the void spaces of the rock media (Rich, 1988). Bouma (1975) found that biological clogging results from growth of bacteria with deposition of polysaccarides and polyurinides, and suspended solids deposition. Rate of accumulation of materials in the pores will depend on the quality of septic tank effluent and the organic loading rate adopted. About the possible nature of failure of rock filter, Rich (1988) inferred that reduced void spaced in mature filters will result in increased velocity of flow (with hydraulic loading rate remaining same) to the extent where critical velocity needed for settling will be exceeded. At this point, the filter will stop removing suspended solids and its performance will deteriorate. It was also concluded that irrespective of the mechanism involved, failure due to clogging can occur much earlier than predicted.

- Water Levels – The filtration process in a rock bed filter is caused by filter materials and root system of plants growing on the filter. Hence, it is necessary to keep a healthy growth of these plants by maintaining the water level in treatment cell and absorption field to about 2 in to 3 in below the rock surface through the bed. Even if there is low wastewater flow from upstream or high evaporation compared to precipitation, such water level in the beds needs to be maintained by adding water from outside. This calls for a regular and more intensive check for water levels and requirement for addition of water as and when necessary (KSU Bulletin, 1998). With performance of the system dependent on such aspect, intense checking and maintenance becomes important on the owner's part.

- Owner's Attention or Participation – In adoption of rock plant filter system for wastewater disposal; owner's participation becomes an important factor. Since growth of plant on rock filters play is instrumental in the successful operation the system, owner's motivation is necessary. The system is said to be best suited to owners who likes to cooperate actively in supporting natural, plant-based disposal system. Rock plant filter unit probably requires more attention and relatively intense effort from the owner than any other disposal system (KSU Bulletin, 1998). In order to ensure proper operation of the system, the owner has to regularly check a number of things. These checks comprise of but are not limited to checking of clear inflow to the rock bed, water level in lined treatment cell, requirement of water for the cells and absorption field, and observe condition of plants in the cells (KSU Bulletin, 1998). As it is difficult to ensure that both present and future owners of the property will have same attitude towards the plant rock filter installed, this particular aspect compromises sound functioning of the system from maintenance perspective.

- Ammonia Removal – Rock plant filters have had a by and large consistent history of problem with regard to removal of ammonia, since its emergence as an effective method of on-site wastewater treatment as available in the literature. This aspect precludes adoption of this treatment and dispersion technique for states and counties having a stringent ammonia limit for effluents (US EPA, 2002). Rock plant filter is almost always preceded by a septic tank where anaerobic form of biological wastewater treatment is applied to reduce the contaminant load from sewage. The septic tank effluent applied on rock plant filter unit on an average contains 40 to 80 mg N/L, of which about 75% is in soluble NH_3-N form and the rest is organic N (Walker et al., 1973; Otis et al., 1974). Sikora and Corey (1976) found that this NH_3-N, if not removed, would ultimately leach into groundwater. This form of nitrogen can later be oxidized into NO_3^- -N form in the long run and drift into ground further to contaminate the groundwater. This is undesirable keeping in view the risks it poses for its use as a source of potable water. Sikora and Corey (1976) confirmed that denitrification is the only mechanism for reduction of effluent nitrogen which indicates that NH_3-N should be oxidized to NO_3^- -N prior to denitrification. Firestone (1982) found out the general requirements imposed by microorganisms for denitrification are, presence of suitable bacteria for producing required enzymes, suitable energy sources to fuel the anabolism, suitable N-compounds to serve as terminal electron sources, and release of oxygen repression of these enzymatic systems. This energy source may be a combination of soil organic matter, C-source present in the effluent, and fresh C-source supplied by the growing vegetation (Stewart and Reneau, 1984). It was inferred that without sufficient energy source, denitrification is not likely to occur (Sikora and Keeney, 1974). Stewart et al. (1979) concluded that soil organic matter might probably be an unsatisfactory denitrification energy source for treating septic tank effluent. As rock is used in the lined compartment of rock plant filter, the influent is deprived of the C-source of soil organic matter that might be critical to drive the denitrification process, even if it is assumed that some extent of nitrification has taken place in the rock bed. Also, in most of the rock filter system design, wastewater gravitates from one unit to another and there is hardly any intermittent dosing sequence practiced. Eastburn and Ritter (1984) concluded that there had been instances where dosing provided a more uniform distribution of effluent and increased denitrification potential of the wastewater disposal system. In rock plant filters there is also a chance that ammonia concentration if the effluent is more than that in the influent (US EPA, 2002). This can be attributed to the anaerobic decomposition of biodegradable fraction of suspended solids captured on the rock surface by filtration process of the unit. Such increase in effluent ammonia content is dependent on weather and shows a seasonal response (US EPA, 2002). As a result, possibility of denitrification to reduce the nitrogen load from wastewater is considered to be slim in case of rock plant filters.

- Residuals – During the operation of rock bed filter, inorganic solids, biological slime layer, and refractory materials will be deposited in the void spaces between the rock surfaces. The rate of such accumulation of materials depends on the amount of biological activity in the bed and also the extent of inert materials making its way into the bed (US EPA, 2002). The frequency of plugging of beds through such deposition can vary. Under present scenario, no provisions for cleaning these surfaces exist. If the plugging problem persists to an extent where it affects normal functioning of the system, it might be necessary to remove the rock media, clear out the accumulated detritus, and replace the entire media (US EPA, 2002). There is no provision of cleaning the slime layers developed over time on the media and disposal of such media also has to be done as per applicable regulations (US EPA, 2002).

Evaluation of System Performance

The basic and immediate use of the information collected in the study is an assessment of the effectiveness of current MDH regulations. Such on-site wastewater treatment operational data collected over a period of time covering most critical wet and cold seasons from a variety of representative sites employing different types of disposal techniques formed a firm basis for evaluation. This also delved into the requirements of any possibility of revising the regulations on the basis of the performance of these on-site wastewater management systems installed. Any modifications required in the design criteria and implementation of these systems has been looked into in light of performance data. The collected field data will add to the records of operational data for different types of on-site wastewater treatment and disposal systems under a variety of environmental settings for reference in future. Such records of operational data can potentially contribute to develop the reliability of these systems under various field settings not only in the state of Mississippi but in other states, too. Any future application of any of these techniques will gain valuable insight into the operation of these systems by taking reference of the collected data. However, use of this operational data has not been restricted to the above and further extended to cover a number of other issues.

Performance data collected over a range of time frame helped enhance the understanding of two very important attributes of on-site wastewater treatment, namely nutrient (nitrogen) control and pathogen (i.e. fecal coliform for the present study) removal. These are the two most critical parameters to adjudge the applicability and effectiveness of any on-site wastewater disposal system from the viewpoint of its compliance to the existing regulations and potential of groundwater contamination. It will be worthwhile to mention that the issue of nutrient control is currently being developed in the state of Mississippi as the authorities are trying to find out suitable numerical criteria for control of nutrient in the states' water body. After the categorization of the prevailing soil component and structure of the sites, the operational data collected has been used in understanding the mechanisms involved in the removal of organic pollutant, nutrient (nitrogen), and pathogens from wastewater. It was most likely that there would be variations in level of contaminant removal for seasonal changes or any other factors. Efforts have been made to define the cause of such variability observed in light of changes and differences in the field conditions. Lastly, comparative analysis is done to look into possible transport of pathogens and organics from disposal fields to the underlying groundwater.

Nitrogen Removal

The issue of nitrogen reduction from wastewater in the disposal techniques employed has been addressed in this research study to supplement and enhance the removal process. It was found from the test results that though reduction of organics from wastewater was by and large significant, some disposal systems were not functioning as effectively as the others in removing nitrogen from wastewater. An attempt has been made to work out certain modifications that might lead to effective nitrogen reduction.

Generally speaking, in the on-site wastewater treatment and disposal systems installed at present, the aspect of nitrogen reduction from wastewater is either absent or expected to be taken care of in the disposal field. The strategy proposed here for nitrogen removal in on-site disposal system, is to introduce a biological nitrogen reduction process prior to its distribution to the field. Such modification can potentially be incorporated in the ATU (aerobic treatment unit) systems installed before disposal. The two possible approaches for this have been simultaneous nitrification-denitrification by operating an activated sludge process (ASP) at low DO concentration and in an intermittent aeration mode. These biological processes are modeled with standard process simulation software package.

Simultaneous Nitrification-Denitrification

Simultaneous nitrification and denitrification (SND) is the process of achieving nitrification and denitrification in a single activated sludge reactor by adopting suitable operating conditions. This has invited particular attention in the past years over conventional systems by virtue of effective nitrogen removal in extended aeration type ASP systems and potential savings in capital and operational cost. For continuously operated plants, nitrogen removal obtained in a single tank can save the cost of a second tank, and low operating dissolved oxygen (DO) requirement can reduce energy cost in maintaining a higher DO level in aeration tank of conventional plants. Such process modifications, if applied effectively, can potentially help meet stringent nitrogen discharge limits.

Biological nitrification is performed in two steps; in the first NH_3-N is oxidized to NO_2^--N (by *Nitrosomonous*), and in the next step it is further oxidized to NO_3^- -N (by *Nitrobactor*). The nitrifying bacteria are autotrophs, and obligate aerobes that use and reduce inorganic carbon in an energy extensive process resulting in low factor of cell synthesis (f_s^o) and biomass yield factor (Y) values. Their chemolithotrophic nature also translates into a small maximum specific growth rate (μ_m) value and large min solids retention time (θ_x^{min}) signifying slow growth of nitrifiers (Rittmann, 2001). Rittmann (2001) also concluded that relatively high value of oxygen half saturation coefficient (K_o) for nitrifying autotrophs signifies that the nitrifiers are not tolerant of low oxygen concentration and continuous operation at an operating DO lower than the K_o value will cause an increase in their θ_x^{min} value resulting in washout of biomass from the system and a consequent rise in effluent NH_3-N concentration. Holman and Wareham (2005) suggested that the initial oxidation of NH_3-N is the rate limiting step of nitrogen removal in SND process. These obligate aerobes are sensitive to dissolved oxygen (DO) concentration and the process of nitrification requires an aerobic environment.

Denitrification is the process in which sequential reduction of NO_3^--N takes place through intermediates like nitrite (NO_2^-), nitric oxide (NO), nitrous oxide (N_2O), and it ultimately produces N_2 gas. In contrast to nitrification, increase in DO concentration greater than even by a few tenths of 1 mg O_2/L can inhibit the activity of the reductase necessary for catalyzing the reactions (Tiedje, 1988; Rittman and Langeland, 1985). Also, very low concentration of electron donor (substrate) and too high concentration of

DO can lead to accumulation of denitrification intermediates, e.g. NO_2^-, NO, and N_2O (Rittman, 2001).

Aerobic heterotrophs responsible for reducing the organic content in wastewater have prolific growth potential. Slower growth rate of nitrifiers can delay the onset of denitrification and denitrifiers can thus eventually face a dearth of organic electron donor to support their metabolism. In other words, there is a possibility that aerobic heterotrophs compete for the same substrate and outnumber the denitrifiers resulting in disruption of denitrification process. Further, oxygen half saturation constant for *Nitrosomonas* is 0.25 – 0.50 mg/L and that for *Nitrobactor* is 0.72 – 2.84 mg/L (Randell et al., 1992) implying that *Nitrobactor* is more sensitive to low DO concentration which might lead to inhibition of second step of NH_3-N oxidation and result in accumulation of NO_3^--N (Holman and Wareham, 2005).

Hence, two apparently conflicting environmental conditions are to be satisfied for simultaneous occurrence of nitrification and denitrification in a single reactor. A few important aspects for onset of SND process can be identified as, (1) operating DO level should not be too low failing to support autotrophic nitrification and also not high enough to inhibit denitrification, (2) sufficient time in the form of SRT or HRT needs to be provided to promote slow growing nitrifiers as nitrification is a prerequisite for denitrification, and (3) adequate electron donor has to be available for heterotrophic denitrification. Rittmann (2001) concluded that that implementation of SND process required the effective combination of solids retention time (SRT), hydraulic retention time (HRT), and DO concentration. So, it is critical to identify the operating conditions so that these two processes, requiring two seemingly different conditions, can occur side by side for effective nitrogen removal.

Further, control of operating conditions of activated sludge process has been implemented with considerable success for improving biological nitrogen removal (BNR) to upgrade existing plants primarily aimed at COD reduction. One of these control parameters has been recognized as DO concentration of in the aeration tank which can be adjusted to optimize nitrogen removal with reduction in operating cost (Lukasse et al., 1998; Copp et al., 2002; Sin et al., 2004; Insel et al., 2006). The fine tuning of operating DO, particularly at low concentration, was observed to be an effective approach for promoting simultaneous nitrification and denitrification resulting in increased nitrogen removal efficiency of the process (Drews et al., 1972; Drews and Greeff, 1973; Applegate et al., 1980; Daigger and Littleton, 2000).

Simultaneous nitrification and denitrification (SND) process of treating domestic wastewater was modeled using Activated Sludge Model no 1 (ASM1). The model is run to identify appropriate operating conditions and interrelationships of three parallel processes of heterotrophic substrate utilization, autotrophic nitrification, and heterotrophic denitrification. The process units considered in this work comprise a complete mix type aeration tank (or reactor) followed by a biomass separator (i.e. clarifier). Further analysis of the process is done to recognize the critical operating parameters and identify their effects on process performance.

Intermittent Aeration Type Activated Sludge Process

Biological nitrogen removal (BNR) from municipal wastewater is increasingly becoming an important issue to limit the problem associated with discharge of treated wastewater having appreciable amount of nitrogen species that can trigger eutrophication in natural water bodies and throw the ecosystem off balance. This has been attempted in various ways over the past few decades, either to retrofit existing wastewater utilities or innovating new operational concepts and methods, e.g. simultaneous nitrification-denitrification (SND) process (Rittmann and Langeland, 1985), nitrogen removal by nitrite pathway (Surmacz-Górska et al., 1997), autotrophic nitrogen removal (Sliekers et al., 2002), etc. In this case, simulation of a conventional activated sludge process (ASP) operated in intermittent aeration mode is performed to alternatively maintain aerobic and anoxic conditions for sequential nitrification and denitrification to occur. Conventionally, this can be done either in a sequencing batch reactor (SBR) or a two stage BNR process having separate anoxic and aerobic reactors in series. In contrast, the intermittent aeration process shows the potential for retrofitting conventional extended aeration type ASP into BNR system, where nitrogen reduction can be achieved in a single reactor saving the cost of a second tank and aeration energy.

A few relevant studies were undertaken to look into the possibility of simultaneous removal of organics (COD) and nitrogen from wastewater by varying the extent of aeration (Bilanovic et al., 1999; Jobbágy et al., 2000; Lukasse and Keesman, 1999; Oh and Silverstein, 1999). Research with full-scale and bench-scale reactors were done by Luzeiro et al. (2002) and Zhao et al. (1998 & 1999), respectively. The extent of removal attainable for different pollutants in intermitted aeration type activated sludge system was investigated by Heduit et al. (1990), Sasaki et al. (1996), and Lucasse and Keesman, (1999). An approach for retrofitting existing plant to intermittent aeration and study of operating parameters had been undertaken by Hanhan et al. (2002), where cycle time and aerobic-anoxic fraction were found to affect the process significantly. Habermeyer and Sánchez (2005) worked on optimization of nitrogen removal in a full-scale biological reactor in terms of on-off period for aeration and DO set point.

In SND process, all three biological processes, namely heterotrophic substrate utilization, autotrophic nitrification, and heterotrophic denitrification, take place concomitantly and carbon reducing heterotrophs and denitrifiers complete for the same organic substrate. Contrary to this, in case of intermittent aeration, the last two processes are separated in time and denitrification occurs in absence of any competition for carbon source from aerobic heterotrophs. For such a system, aerobic and anoxic conditions are created in the aeration tank by periodically turning the air supply on and off. In the aerobic phase, autotrophic nitrification is inflicted by conversion of influent NH_3-N (and that available from previous anoxic phase) to NO_3^- -N, with heterotrophic reduction of organic carbon occurring in parallel. In anoxic phase, NO_3^- -N acts as electron acceptor and is reduced to N_2 gas by denitrification. Organic carbon entering with the influent (and that remaining at the end of aerobic cycle) provides the necessary electron donor for heterotrophic nitrogen removal, while aerobic COD reduction remains suppressed. The combination of aerobic and anoxic phase constitutes cycle time (CT) and the ratio of anoxic time period over cycle time is termed as anoxic time fraction (AF).

Sequential nitrification and denitrification process by intermittent aeration has been modeled using Activated Sludge Model no 1 (ASM1) that incorporates 7 heterotrophic, 5 autotrophic, 3 hydrolysis, and other kinetic and stoichiometric coefficients. The work is aimed at identifying the operating parameters that affect process performance significantly, and finding out their ranges for optimum nitrogen removal.

Keeping in mind the advantages of aerobic treatment units in on-site treatment in comparison to septic tanks and also the fact that about two-thirds of all land area in United States is unsuitable for installation of septic systems (Linsley, 1992), modifications have been suggested for the aerobic treatment units (ATU) to complement nitrogen reduction. Also, it is well documented that nitrification and denitrification rates in the soil profile have been found to be better when effluent was applied in a brief daily pulse rather than continuous (Beggs et al., 2004). In all the on-site regulations referred to earlier for various disposal techniques, it was categorically mentioned that the dose of effluent from treatment unit over the field has to be intermittent and not continuous. This is mentioned in order to facilitate nitrogen removal in the disposal fields by alternatively creating anoxic (in effluent application cycle) and aerobic (over resting cycle) conditions for denitrification and nitrification in the soil matrix, respectively. Therefore, it is perceived that the same strategy, if implemented in the ATU by aerating it intermittently, might enhance nitrogen reduction in the treatment unit before its disposal in the field.

A description of these two process simulations with relevant considerations and assumptions is given in the next chapter. It was intended that the simulation results obtained would be used to gather important information culminating into proposed modifications in the ATU to enhance nitrogen reduction to an appreciable extent. Simulations performed for identifying the operating conditions of establishing such process was performed by considering a conventional activated sludge plant that has been extrapolated for its applicability to on-site treatment units.

CHAPTER IV

MATERIALS AND METHODS

This chapter outlines the materials and methods used and experimental design. Methods for the performance evaluation of on-site wastewater disposal techniques are then provided followed by methods for evaluation of proposed modifications in treatment system for nitrogen removal. This includes specific details of major tasks undertaken and the experimental design followed covering all aspects of identification of sites, procedure of installing monitoring stations, field sampling and testing, generating performance evaluation data, biological process modeling and simulation, and ultimately culminating into interpretation of results and discussion. The salient points relevant to the study methodology are given in the following.

Identification of Representative Sites

The basic criteria used for site selection was that the sites should be operational for more than 5 years but less than 15 years. This was proposed to ensure that the units selected were not new but matured, and also not too old and outdated. Adopting this selection criteria ensured that the systems were well established in their field settings and would represent a relatively steady state of operation for proper evaluation. The sites were all residential dwellings using on-site treatment and disposal for wastewater management.

Other selection criteria were, these being representative of the diversity of sub-surface morphology, use of types of on-site wastewater disposal techniques presently permitted in the area, and owner's consent to take part in the study. Out of the total six sites selected, two each represented one of the three types of disposal systems being evaluated. The addresses and locations of these sites are given in Appendix A. These six sites were located in Harrison and Jackson counties along the Mississippi gulf coast (four in Ocean Springs, one in Biloxi, and one in Gulf Port). Specific information about the dwelling unit and the on-site disposal system installed were collected by a site survey questionnaire (Appendix B) given to the inhabitants of these dwelling units. A summary of information collected by this survey is given in Appendix C. The different types of systems included in the study were:

- Septic tank with Subsurface Drip Irrigation System (Leaching fields),
- Aerobic Treatment Unit (ATU) with Spray Irrigation Disposal System, and
- Septic tank with Elevated Sand Mound Disposal System.

Site Characterization

Soil sampling and analysis was done at these designated sites to ascertain the sub-surface morphology. Such information was helpful in establishing the type of soil and its matrix in terms of probable contaminant removal processes occurring in different wastewater disposal techniques installed. No restrictive layer was found within a depth of 6 ft from ground surface in any of these sites. Characterization of sites in terms of existing soil included soil sampling, sample preservation, testing, analyses, and characterization.

Soil Sampling

Soil samples were collected at monitoring well locations at every 2 ft of depth. The reason behind collection of soil samples at a reasonable interval of 2 ft was that no sudden change in sub-soil conditions could be seen at these locations. As the monitoring wells were installed at 6 ft depth, three soil samples were collected from 2 ft, 4 ft, and 6 ft of depth at each monitoring well location. The number of samples from each site (with 4 monitoring wells) was 12 and from all the six sites were 72. The soil samples were secured and preserved at 4^{o} C temperature until tested in the laboratory.

Soil Testing

Testing and characterization of soil was taken up by analyzing the soil samples for the following parameters. The test procedures adopted were:

- Water Content – This was the first test performed on samples as per ASTM D 2216-98.
- Liquid Limit (LL) – Test specimens were prepared with the "Dry Preparation Method" as shown in ASTM D 4318, Section 10.2 and dried at room temperature. Liquid limit was determined using "Method A" (multipoint LL) shown ASTM D 4318, Section 11.
- Plastic Limit (PL) – Test specimen was prepared according to Section 15 of ASTM D 4318, and procedure used was "Hand Method" as per Section 16.1, 16.2.1, and 16.3. The test was repeated according to Sections 16.5 and 16.6 with the calculations done as per Section 17.
- Plasticity Index (PI) – Calculations performed as given in Section 18 of ASTM 4318.

- Particle Size Distribution – About 120 gm of test specimen was dried and weighed according to Section 6 of ASTM D 1140 and sample was prepared as per Method B of Section 7.3.1. Sample was washed for at least 20 min over # 200 sieve to ensure all the fines were washed out. The retained material was dried and weighed. The dried material was then shaken in a shaker for 5 min through a series of sieves consisting of ¾ in, 3/8 in, # 4, # 8, # 16, # 30, # 50, # 100, and # 200. Weight of materials retained on each sieve was recoded.

- Porosity – A graduated cylinder was filled with 50 ml of water. Soil sample was poured and mixed thoroughly with the water and allowed to settle for at least 8 hrs. The displacement in water volume was recorded to find the porosity of soil.

In conjunction with the above, supporting laboratory calculations were done to find the hydraulic conductivity and complete taxonomic characterization of soil samples was done. Laboratory analyses were performed by Soil Testing Laboratory of Civil & Environmental Engineering Department of MSU in accordance with the QAPP earlier formulated for the project and applicable QA/QC procedures. Hydraulic conductivity of soil was calculated from the grain size distribution analysis by the following formulae, (www.Groundwatersoftware.com)

$$K = \frac{\rho g}{\mu} 8.3x10^{-3} \frac{\eta^3}{(1-\eta)^2} d_{10}{}^2 \qquad \text{Eqn 1}$$

Where, ρ is density of water at 20° C in kg /m^3, g is acceleration due to gravity in m^2/s, μ is dynamic viscosity of water at 20° C in N-sec/m^2, "$8.3x10^{-3}$" is unit conversion factor, η is porosity of soil, d_{10} is rain size that is 10% finer by weight (effective size), and K is estimated hydraulic conductivity of soil in m/sec. The value of K is ultimately expressed in cm/sec.

Soil Classification

Classification of soil was done from the Soil Texture Triangle (Appendix D) to categorize sand and silt, following particle size scale proposed by the USDA. Further classification of fine-grained soils has been performed based on the Plasticity chart as furnished in Appendix E (Sowers, 1970).

Lysimeter Locations

Six (6) lysimeters were placed at every disposal site, each in a separate hole. These were installed at three appropriate locations in the field in three groups each having two lysimeters located close to each other. Different locations in the disposal field were selected so as to monitor possible variation in the level of treatment rendered on percolating wastewater by the installed disposal technique. Lateral spacing between adjacent soil water samplers was about 12 in to minimize possible influence of adjacent samplers during sampling. It was decided that monitoring of contaminant concentrations at levels closer to the ground water table might be a better way to evaluate the scenario of contaminant migration in contrast to looking at this aspect at higher levels of soil strata with respect to the subsurface water table. In other words, the contaminants detected at the trench bottom level had still got the chance of undergoing further treatment as it percolated further down into lower soil strata. Whereas, the contaminants detected at the lower levels (i.e. at compliance depth and 1 foot below it) were already close to reaching the water table and had got relatively less distance to travel down and consequently less chance to undergo further treatment by soil strata.

Hence, to detect the critical concentration, it would be better to monitor the level of pollutants at these lower soil strata instead of monitoring it in the upper one. If the level of contaminants found in the lower two levels were satisfactory when compared against the discharge limits, one could be in a position to infer about the adequacy of that particular disposal system.

The lysimeter selected was Pressure-Vacuum Soil Water Sampler (Model 1920F1) made of a 36 in long, 1.9 in outer diameter PVC tube (made of FDA-approved material) with a 2 bar ceramic cup bonded together, supplied by Soilmoisture Equipment Corporation, California, USA. The locations of lysimeters with respect to layout of the disposal system are summarized below and shown on the layouts (Appendix F.1 through 6).

- Sub-surface drip irrigation – The three groups were placed at three separate locations along the length and profile of this distribution pipeline; the first set at approximately 20% of total pipe length from the point of discharge, the second one at halfway down the length, and the remaining one at 80% of total pipe length.

- Sprinkler system – The two such sites selected each had a three-nozzle type system; yet one had sprayers distributing the effluent in one direction (site 3) and the other one was a rotating-type (site 4). For the first type, three locations picked were just inside the disposal field laterally equidistant from each of the three sprayers. For the rotating type system, the three locations selected were, two at the contact point of two adjacent sprinklers' influence circles, and one at the edge of the middle circle inside the disposal field.

- Mound system – With the site conditions permitting, the three locations were selected around the elliptical mound to capture the wastewater characteristics that were migrating in all directions from the mound (Site 6). Otherwise, these locations were taken at three different places based on the predicted path of flow of wastewater from the mound (Site 5). The samplers were not installed on the mound so as not to undermine the proper functioning of the system.

Depth of Lysimeter

As per the existing regulations, there should be a separation of 2 ft between the bottom of trench or disposal field and the seasonal groundwater table. Difficulty in assessing the minimum depth of ground water table in these sites required adoption of top of groundwater table as compliance depth in placing the lysimeters vertically. The pipes in drip irrigation fields were located by and large within 1 foot of ground surface. Considering another foot as the trench bottom and its required separation of 2 ft from groundwater table, the two lysimeters were placed at 4 ft and 5 ft for the sub-surface drip irrigation type system. For sprinkler and mound systems, wastewater was dispersed over ground surface and lysimeters were placed at 3 ft and 4 ft depths.

Monitoring Well Locations

Four (4) groundwater monitoring wells were installed on each site at the four corners of the demarcated disposal field or the site boundary depending on field conditions. The wells were installed at a 6 ft depth with 1.5 ft of (1 mm) screen at the bottom. Arrangement of the monitoring wells was such that the well positions formed a "bounding box" encompassing the on-site wastewater disposal field. Separation between the monitoring well and disposal system was adequately maintained to mitigate possibility of well contamination. The wells were made of 4 ft long, 2 in inside diameter PVC tube (made of FDA-approved material) clumped with a 1.5 ft screen having a screwed on tip and supplied by Soilmoisture Equipment Corporation, California, USA. Figures given in Appendices F.1 through 6 illustrate the approximate locations of groundwater monitoring wells with respect to the on-site wastewater disposal systems at various sites.

Equipment Installation

Lysimeters were installed by first digging a 6 in diameter hole to appropriate depth at specific location in the ground with a diesel operated hand-held auger. The 200 mesh silica slurry (from manufacturer) was poured first into the hole in adequate quantity to cover the ceramic cup of the lysimeter entirely. This slurry would help draw water towards the cup when vacuum was created inside the lysimeter during sampling. Placing and holding the lysimeter in position, the hole was backfilled up to 1 foot from the ground level. A 6 in thick bentonite (supplied by manufacturer) plug was provided over this backfilling to avoid intrusion of surface water into the hole. The remaining top 6 in of the hole was packed with backfill material.

For installing monitoring wells, a 6 in diameter hole was dug to stipulated depth the same way as the lysimeter. The monitoring well assembly was fabricated at the field with a screwed-on top conical section, and a 1 and ½ ft screen tightly clamped with an appropriate length of monitoring well. The assembly was then lowered, held in position and sufficient quantity of ordinary sand was poured into the hole to fully cover the screen to permit water inside the well. A 6 in thick bentonite plug was provided over this sand before backfilling the hole up to the ground level. The lysimeters and monitoring wells, after installation, were each provided with a PVC cover with a screwed-on top and lock and key to secure the equipment.

Sampling Period

The sampling of wastewater from different sites was done over a period of 6 months starting from mid-November 2008 to mid-May 2009. The period selected covered wet-dry and warm-cold months of the year in order to have an overall perspective of possible variation in degree of treatment received by the wastewater in these disposal fields.

Sample Collection

Sample collection from each site was done following a specific schedule. Samples were collected from each site on every 16^{th} day depending on weather. Three sites were sampled on every trip made, and the other three were covered in the next trip. Over the sampling period, a total no of 21 sampling trips could be made to the project area with a few being cancelled due to rain. Simultaneous sampling of adjacent lysimeters was done to minimize short-circuiting or cross-contamination of samples being pulled from separate depths. Samples from lysimeters were collected by creating and maintaining a vacuum of 15 in to 20 in Hg by a vacuum pump for a period of 1 to 1 and ½ hr at the maximum. Quite understandably though, it took much less time to pull samples when the fields were wet. Samples from monitoring wells were collected by bailers. First the accumulated water in wells was bailed out and the wells were allowed the recharge. Fresh water samples were collected once the wells recharged. A few of the wells did not recharge well over the time spent for sampling. The initial samples collected were retained for such wells.

Sample Testing

The list of parameters analyzed for wastewater and groundwater samples were Chemical Oxygen Demand (COD), 5-day Biochemical Oxygen Demand (BOD), Total Kjeldahl Nitrogen (TKN), Ammonia-Nitrogen (NH_3-N), and Fecal Coliform (FC). The procedures for analysis of parameters are given in Table 4.1. Samples were collected, preserved suitably at the site and brought back to laboratory for testing. However, only the fecal coliform samples were filtered and incubated at the site or within 6 hours of sample collection as stipulated. For TKN and Ammonia tests, some results were found to be less than the "blank" value, these have been demarcated as "below detection limit" (bdl). Subsequent statistical analysis with these results was done considering these values as "0". Concentrations, that were found to be lower than the least concentration that could be measured by any particular test, were marked as "below sensitivity limit" (bsl).

Sample Dilution

The amount of sample required from each equipment location was about 400 to 500 ml if all of these tests were to be run without dilution. However, there had been instances where sufficient sample could not be extracted from some of the monitoring stations to run all the tests as the disposal fields were found to be dry. Suitable dilution factors were used to test the parameters whenever low sample volumes were obtained. The samples were appropriately diluted so that the resulting concentration would not fall beyond the two extremities of the standard curve prepared for a couple of the tests or below the detection limit for the test.

Table 4.1

Proposed List of Water Quality Sampling Parameters and Methods

Parameter	EPA Analytical Method [1]	Preservation	Holding Times Recommended or Regulatory Maximum	Detection Limits
5-day Biochemical Oxygen Demand (BOD)	405.1	Refrigerate at 4^{o} C.	6 hours/48 hours	2 mg/L
Chemical Oxygen Demand (COD)	410.4	Analyze immediately, or add H_2SO_4 to pH <2 and refrigerate at 4^{o} C.	7 days/28 days	2 mg/L
Ammonia, Nitrogen (NH_3-N)	350.3	Analyze immediately, or add H_2SO_4 to pH <2 and refrigerate at 4^{o} C.	7 days/28 days	0.1 mg/L
Total Kjeldahl Nitrogen (TKN)	351.3	Add H_2SO_4 to pH <2 and refrigerate at 4^{o} C	7 days/28 days	0.1 mg/L
Fecal Coliform (FC)	SM [a] 9222 D	Collect in sterile containers and refrigerate	6 hours	10 org/ 100 ml

[a] SM = Standard Methods (APHA, 1989).

Prioritization of Testing

On collection of insufficient volumes of samples, prioritization of tests was done when it was perceived that adopting too high a dilution factor might invalidate the test results. In such cases, COD and TKN tests were initially given preferences over other tests as they would require less volume of sample. After a reasonable amount of data was collected for these parameters, the focus of testing was then shifted to the remaining tests (e.g. FC, BOD, and NH_3-N) in order to cover the entire range of parameters selected.

Discharge Limits

Performance evaluation of disposal techniques would have to be done against some stipulated discharge limit or numeric values for various contaminants. Neither the MDH nor MDEQ have any available performance standards or regulations specifically meant for on-site wastewater disposal. Suitable values of the main wastewater constituents (BOD and fecal coliforms) were adopted as those for surface water discharge from a conventional treatment plant. The discharge limit selected were 30 m/L BOD (5 day at 20^0 C), 5 mg/L of TKN, and 200 cfu/100 ml fecal coliforms. The same limit was also been applied to investigate any migration of pollutant from the disposal field to groundwater.

Data Compilation, Correlation and Analysis

The field data collected from the sites comprised of water levels at monitoring wells and ambient air temperature during sampling. The sample testing results included concentrations of contaminants, e.g. COD, BOD, TKN, NH_3-N, and fecal coliforms, at six lysimeter and four monitoring well locations at each site. Results for each site were summarized and presented appropriately for subsequent comparison and performance evaluation. Historic weather data on precipitation and temperature had been collected over the data collection phase to identify rainfall event and define cold and warm season.

Data Segregation

Apart from performance evaluation of various disposal techniques, it was required to assess the extent of variation in level of treatment received by the wastewater as a result of seasonal changes. Accordingly, it was necessary to segregate the results as dry-wet and cold-warm weather data and perform suitable statistical analysis to conclude whether such variation in weather significantly affected the contaminant concentrations or not. The following criterion was used for segregating the results under the two separate categories mentioned above:

- Warm and cold weather data – An ambient temperature of 54^0 F (approximately 12^0 C) was selected as the demarcating temperature to separate warm and cold weather data. Data collected when the ambient air temperature was at or more than this, had been categorized as warm weather data; and that collected at less than this was considered cold weather data.
- Wet and dry weather data – The data collected for a particular pollutant was arranged according to increasing depth of observed water level from one the four ground in monitoring wells which was closest to the disposal field and closely represented the condition of water level in that field. Segregating the data at any particular depth and comparing the statistical significance of the mean values through suitable statistical technique, the probability of data varying significantly was calculated. This method was repeated for different observed water depths in the wells. The observed depth that indicated the highest probability that the means of wet and dry data of that particular pollutant would vary significantly was used to segregate wet and dry weather data.

Data collected from all the monitoring stations in any particular disposal field (i.e. 6 lysimeters and 4 monitoring wells) were categorized as per the criteria described above. After segregation of data for any given site, statistical analysis on the data sets was performed for assessment of performance for that disposal system.

Groundwater Flow Direction

Besides performance evaluation, the potential of these disposal techniques for possible groundwater contamination was evaluated. The direction of groundwater flow in each disposal field was established to identify if any pollutant load being contributed by the disposal field across which movement of groundwater was taking place. To monitor this, water samples were collected from 4 monitoring wells installed in each site and tested for the same parameters as mentioned earlier.

Groundwater elevations in each of these wells (4 in each site) were recorded during every sampling. Based on the water level data collected, the possible direction of groundwater flow across the disposal field was identified with the criteria described below. The highest and lowest water levels and the corresponding wells were identified.

- If, it was observed that these identified wells show consistency in collected data as the highest and lowest water elevations for that particular site, then the direction of groundwater flow was ascertained accordingly.
- Some of the data collected was not found to be consistent and varied over the sampling period. In such cases, the largest number of observations in any two wells that consistently were identified as the highest and lowest water levels indicated the most probable direction of groundwater flow.

On having established the direction of groundwater flow across disposal field by this method, statistical analysis of water quality data sets collected from the upstream and downstream monitoring wells was done to look into the possibility of groundwater contamination and probable pollutant migration from the fields.

Depth-wise Variation in Treatment

The basic purpose of installing 6 lysimeters at two different depths in the disposal field was to determine any variation in treatment received by the percolating wastewater. These two selected depths were compliance depth, as explained earlier, and 1 foot below this depth. In other words, it was investigated under this project if addition of an extra foot to the existing compliance depth might prove to be beneficial for any or all of these disposal systems in meeting the discharge limits. For any given site, the two sets of data collected, one at shallower and the other at deeper level on concentrations of various pollutants, were statistically compared to each other to draw relevant conclusion.

Statistical Analysis

Statistical techniques had been applied to analyze the test results and draw conclusions on their performance. The basic assumption behind performing such statistical analysis was that, since similar types of residential dwellings were selected, the quality of influent coming out of these households did not vary over wide range and were by and large comparable, so that application of such mathematical tools were valid and did not affect such analysis in a significant way. The various techniques used in this regard are briefly described below.

Average Concentration

The average concentrations and standard deviations of the tested parameters at different locations in each of the sites were computed using standard equations to present an overall scenario of level of treatment achieved by the disposal techniques and its variations over time. Similar calculations were performed on data sets categorized in terms of weather conditions, such as cold, warm, dry, and wet.

C_{95} Concentration

Assuming standard normal distribution of contaminant concentration, C_{95} concentration of any parameter could be defined as the concentration, 95% of the values of concentrations of that particular parameter would fall either at or below that value.

With the sample mean and standard deviation values available from above, such concentrations were computed using "t-distribution" for the contaminants. This analysis was performed for the entire data set and then for data sets categorized on the basis of weather conditions. Assessment of different disposal techniques was done by comparing these C_{95} concentrations against adopted discharge limits.

ANOVA Analysis

The "analysis of variance" or ANOVA technique was used to find if there is any statistically significant difference between any two or more data sets. Single factor ANOVA could be applied to test the hypothesis to check if means of two different treatment levels were equal, (in which case τ_1 and τ_2 are taken as the mean values).

For such a single factor ANOVA test, a null hypothesis was assumed as H_o: $\tau_1 = \tau_2$ and the alternative hypothesis is adopted as H_1: $\tau_1 \neq \tau_2$. The two data sets were considered as having two separate normal distributions with their respective mean and standard deviation values. In testing the above hypothesis, the alpha level (α) was not calculated but selected as appropriate. In the present context, considering a nominal acceptable analytical error of 5%, α-value of 0.05 was adopted. Once the alpha level had been set, a test statistic was computed. Each statistic had an associated probability value, called the "p-value". This is the likelihood that there is no difference between the data groups. Therefore, if the p-value was greater than α (0.05), one could assume that the two conditions (e.g. wet vs. dry, cold vs. warm) did not produce different levels of treatment.

This technique was used to assess if weather conditions affected treatment, any depth-wise variability in extent of treatment, and potential migration of pollutant to groundwater by comparing the collected data sets, as applicable. With the sample sizes, means, variances, and data categorized in terms of depth, direction of groundwater flow, and weather conditions, hypothesis of equal mean of concentrations of any pollutant was tested using either Statistical Analysis Software (SAS, North Carolina) or MINITAB (Pennsylvania) program. The types of analysis applied for such evaluation is given as under:

- Effect of dry vs. wet weather condition – single factor ANOVA,
- Effect of warm vs. cold weather condition – single factor ANOVA,
- Effect of interaction of weather conditions – multiple factor ANOVA,
- Effect of depth on treatment – single factor ANOVA.
- Effect of treatment on groundwater quality – single factor ANOVA
- Effect of disposal technique on treatment – single factor ANOVA

Important inferences were drawn based on such analysis to ascertain the performance of these disposal techniques and draw conclusions on other relevant issues investigated as given in the next chapter.

Biological Process Simulation

The two possible approaches for nitrogen removal in on-site wastewater system attempted under the study were simultaneous nitrification-denitrification by operating a conventional activated sludge process at low DO concentration or on an intermittent aeration mode. Such processes had been simulated with a standard process simulation software package GPS-X (Hydromantis, Inc., Hamilton, Ontario) version 5.0. These two process simulation and relevant considerations are given in the following sections.

SND Process Simulation

Simultaneous nitrification and denitrification (SND) process of treating domestic wastewater was modeled using Activated Sludge Model no 1 (ASM1). The model was initially run to recognize appropriate operating conditions and interrelationships of three parallel processes of heterotrophic substrate utilization, autotrophic nitrification, and heterotrophic denitrification. The process units considered in this work comprised a completely mix aeration tank (or reactor) followed by a biomass separator (i.e. clarifier). After identification of the operating window for SND process, the effects of other important factors like BCOD:TKN ratio, hydraulic retention time (HRT), and recycle ratio (R) on the process was investigated. This SND process modeling incorporated 7 heterotrophic, 5 autotrophic, 3 hydrolysis, and other kinetic and stoichiometric coefficients. All the process simulations were performed by adopting values of these parameters as recommended by Cox (2004), given in Appendix G. Simulations of simultaneous nitrification and denitrification (SND) process were done by GPS-X, a simulation package that includes ASM1 modeling. The work was done in two phases. Typical influent characteristics of influent wastewater were adopted as given in Table 4.2.

Table 4.2

Influent Characteristics [a]

Component[b]	ASM1 Symbol	Concentration [c], mg/L
Soluble inert organic material	S_I	0
Readily biodegradable substrate	S_S	160
Particulate inert organic material	X_I	30
Slowly biodegradable substrate	X_S	240
Non-biodegradable particulates from cell decay	X_D	0
Free and unionized ammonia	S_{NH}	25
Soluble biodegradable organic nitrogen	S_{ND}	6.5
Particulate biodegradable organic nitrogen	X_{ND}	8.5
Nitrate and nitrite	S_{NO}	0

[a] The same influent characteristics were also considered for intermittent aeration type system.

[b] Typical values (Grady et al., 1999). Active biomass was absent in the influent.

[c] Expressed as COD for organics, and as N for various nitrogen species.

In the first set of simulations, the model was run taking influent characteristics (Table 4.2), mean values of the 19 kinetic and stoichiometric parameters (Appendix G) over a range of solids retention time (SRT) values (Table 4.3) for particular operating dissolved oxygen (DO) concentration. Then this process was repeated for a series of operating DO levels varying from 0.1 mg/L to 2.0 mg/L. Applicable operating parameters (i.e. DO concentration, and SRT) for occurrence of SND process and effective nitrogen removal were identified from these runs. Adopting the same influent organic concentration, a second set of simulations were run for the selected combination of operating DO and SRT to identify the effects of other parameters [ratio of biodegradable COD to total kjeldahl nitrogen (BCOD:TKN), hydraulic retention time (HRT), and recycle ratio (R)] on total nitrogen removal. Appropriate values of such operating parameters were then selected for the last set of simulations. The range of values of operating parameters considered in these simulations to indicate the operating window for onset of SND process is furnished below (Table 4.3).

The simulation performed for SND process with the selected and default values of operating parameters (as 0.4 mg/L DO concentration, 15-day SRT, 12-hr HRT, 0.5 recycle ratio, and an influent BCOD: TKN ratio of 10) and mean values of kinetic and stoichiometric parameters (Appendix G) was termed as discrete simulation.

Table 4.3

Process Operating Conditions for SND Simulations

Simulations	Parameters [a]	Range examined [b]	Process Configuration
Phase 1	DO	0.10 to 2.0 mg/L	Complete mix aeration basin (with sludge recirculation and wasting) and secondary clarifier
	θ_X	1 to 30 days	
Phase 2	θ	4 to 24 hr	
	R	0.25 to 3.0	
	BCOD:TKN	4 to 20	

[a] Symbols: DO – dissolved oxygen concentration in aeration tank, θ_X – solids residence time (SRT), θ – hydraulic retention time (HRT), and R – sludge recycle ratio (R)

[b] Selected ranges typical of a range of SND process configuration (Rittman, 2001)

Stochastic analysis was done for 1,000 Monte Carlo simulations of combinations of values of 15 parameters taking the same operating parameters as above. Model simulations were run to obtain the PDFs of steady state effluent concentrations of various nitrogen species and COD. The results were analyzed to examine the sensitivity of SND performance on selected dependent variables (e.g. S_S, S_{NH}, S_{NO}, S_{TN}), and sensitivity of overall nitrogen removal on selected model parameters or independent variables (as mentioned above). It was aimed at assessing the reliability of the SND process from the viewpoint of variation in values of these process parameters. Lastly, the output results of these simulations were tested by computing the Spearman rank correlation test using Statistical Analysis Software (SAS, North Carolina) for each of these 15 parameters to identify those out the entire set which most effectively influenced overall nitrogen removal.

Intermittent Aeration Type Activated Sludge Process Simulation

Biological nitrogen removal by intermittent aeration (IA) in a continuously mixed activated sludge reactor was simulated using the same model GPS-X. Wastewater mixing and aeration is taken as volumetric kW input (kW/10^3 m^3) considering appropriate default values of associated parameters (e.g. oxygen transfer rate, "α", "β" factors, etc.).

Typical characteristics of influent wastewater were adopted (Table 4.2). In the initial set of simulations, a cycle time of 3 hr was adopted and change in nitrogen removal is recorded over anoxic time fraction varying from 0.1 to 0.9. The power input to the system was taken as the third variable varying from 10 kW/10^3 m^3, the minimum to ensure completely mixed flow regime (Rittman, 2001) to 90 kW/10^3 m^3, beyond which shearing of flocs occurred (Grady et al., 1999) on the basis of per 1,000 m^3 of tank volume. Possible ranges of operating variables, as identified from these results, were used for running the next set of simulations. The oxygen uptake rates of biomass were computed considering the same model operating in a conventional activated sludge mode at 2 mg/L DO level (Table 4.5), and accordingly power input to the reactor at different SRTs is calculated by the following formulae (Grady et al., 1999),

$$P = \frac{RO}{\eta_P} \qquad \text{Eqn. 2}$$

Where, P was the power input in KW, RO was the oxygen requirement in kg/hr, and η_P was the in-process energy efficiency for the mechanical aeration system in kg O_2/kW/hr.

Table 4.4

Process Operating Conditions for IA Simulations

Operating Parameters [a]	Range examined [b]	Default value adopted	Process Configuration
Cycle time (CT)	1 to 24 hr	3 hr	Completely mixed aeration basin (with sludge recirculation and wasting) and secondary clarifier
Anoxic time fraction (AF)	0.10 to 0.90	0.45	
θ	6 to 24 hr	12 hr	
R	0.25 to 3.0	0.5	
θ_X	15 to 25 day	15, 20, 25 day	

[a] Symbols: θ_X – solids residence time (SRT), θ – hydraulic retention time (HRT), and R – sludge recycle ratio (R)

[b] Selected ranges typical of a range of intermittent aeration type nitrification-denitrification process

Table 4.5

Power Input Requirement for Surface Aeration

Description	SRT values adopted, day				
	5	10	15	20	25
Oxygen uptake rate for aerobic system, mg O_2/L/day	642.36	747.35	830.42	837.50	861.43
Oxygen requirement (RO), kg/day [a]	13.38	15.57	17.30	17.45	17.95
Required power input, kW [b]	19.11	22.24	24.71	24.93	25.64

[a] calculated by multiplying oxygen uptake rate with the volume of reactor taken as 500 m^3
[b] η_P adopted is 0.7 kg O_2/kW/hr (Grady et al., 1999)

These power inputs were subsequently used for corresponding values of SRT in the next set of simulations performed. Ranges of operating variables, i.e. cycle time (CT), anoxic time fraction (AF), hydraulic retention time (HRT), recycle ratio (R), and solids residence time (SRT), over which the process performance was evaluated, is given in Table 4.4. The default values of these parameters were selected based on results of first set of simulations. Effect of these operating parameters on the process was studied by changing these in turn for three different SRT values (15, 20, and 25-day).

A similar analysis was done for diffused aeration type system where aeration and mixing were controlled by volumetric air flow input into the reactor (i.e. $m^3/min/10^3\ m^3$). The oxygen uptake rates of biomass were calculated as earlier considering the process being operated at 2 mg/L DO level in conventional activated sludge process (Table 4.6), and accordingly air flow requirement to the reactor at different SRTs were found by the following formulae (Grady et al., 1999),

$$Q = \frac{6.0RO}{\eta_Q} \quad \text{Eqn. 3}$$

Where, Q was the air flow rate in m^3/min, RO was the oxygen requirement in kg/hr, and η_Q was the field oxygen transfer efficiency expressed as % of oxygen in the air actually transferred to the liquid.

Table 4.6

Air Flow Requirement for Diffused Aeration

Description	SRT values adopted, day				
	5	10	15	20	25
Oxygen uptake rate for aerobic system, mg O_2/L/day	642.36	747.35	830.42	837.50	861.43
Oxygen requirement (RO), kg/day [a]	13.38	15.57	17.30	17.45	17.95
Required oxygen input, m^3/min [b]	1,338	1,557	1,730	1,745	1,795
Required air flow input, $m^3/min/10^3\ m^3$ (assuming 21% O_2 in air)	12.74	14.83	16.48	16.61	17.10

[a] Calculated by multiplying oxygen uptake rate with the volume of reactor taken as 500 m^3
[b] η_P adopted is 0.06 (Grady et al., 1999)

The concentrations of various wastewater constituents reported here were averaged over the length of the cycle time that they correspond to and the simulations had been run adopting mean values of kinetic and stoichiometric parameters as proposed by Cox (2004) (Appendix G). In case of both simulations, the secondary clarifier was modeled as an ideal biomass separator and sludge wasting was done from aeration tank to maintain the stipulated SRT.

CHAPTER V

SYSTEM PERFORMANCE: ANALYSIS AND EVALUATION

This chapter describes analysis of all field data collected and compiled in this study, statistical analysis of results from different perspectives, and relevant inferences drawn in line with the objectives of the project. Important information was extracted from data analysis that comprised performance assessment for disposal techniques, variation in performance under different weather conditions, possible transport of contaminants to groundwater, etc. These are presented in the following sub-sections. Though originally included, field performance evaluation of rock plant filter type disposal technique could not be undertaken. The immediately following section delineates the justifications for excluding rock plant filter type disposal technique from present scope of research.

Exclusion of Rock Plant Filters

Visits were made to a number of residential dwelling units along the coast using rock plant filters for on-site wastewater disposal. Yet, not a single unit was observed to be working adequately enough so that it could be included in this evaluation. Most of the sites visited had a similar problem of wastewater surcharge to the ground surface. This could be traced back to plugging of pores in rock primarily with plant rooting and deposition of organic materials over time on rock surface. As per the design requirements, roots of the plants growing over the rock filter were not expected to stretch beyond the top 1 foot to 2 ft of media depth. However, it was observed that the rooting system of the plants selected and grown on the installed filters was covering the entire depth of media plugging the pores and even the filter inlet and outlet. This conflicted with the flow path of water in and out of filter and was causing the surcharge problem. In addition, there was problem of improper selection of plants that had a longer root network clogging the filter pores undermining system performance. In some cases, the plants were not being maintained properly, affecting root growth and in turn compromising the expected filtering of wastewater by root system. It could be inferred that maintaining adequate plant growth over rock filter was mandatory for such kind of disposal technique. It became somewhat exhaustive on the part of the owner, unlike and more than other available systems, to check certain things about rock plant filter in order to have it operating properly.

Accordingly, rock plant filters were excluded from the present study. Apart from the above, there were other issues or shortcoming involved with this type of system as discussed earlier, such as poor ammonia removal, inconsistent effluent quality, no provision for media cleaning, media replacement, and owner's attention. It was concluded that such type of disposal technique might not be considered a feasible option for on-site wastewater management in the Mississippi coastal areas.

Site Selection

Selection of appropriate and representative sites for the project along the coast had to depend primarily on MDH's involvement in the process and owner's consent.

Initial selection of sites was done at the start of the project back in 2005, but things took a different shape after hurricane Katrina hit the Mississippi coast in August 2005. The project was held up for a couple of years in the aftermath of the catastrophe and a few of the selected sites were rendered unusable after the devastation caused by the hurricane. After the project was restarted in the middle of 2008, a few new sites were identified by the MDH to compensate for the unusable sites and a fresh set of negotiations had to be taken up with the owners for their approval.

Although the drip and sprinkler irrigation sites were identified relatively readily with owner's consent received, finding sites with mound system took a while. The project could thus be started in late September 2008, while search for the last site was still being pursued with the MDH. Eventually a suitable site was found, but this whole process delayed securing of all six sites. Installation of equipment in the six sites was completed by early November and sampling started from middle of November 2008.

Analysis of Test Results

The entire set of data collected for the study from all the monitoring stations for six sites is furnished in Appendices H.1 through 6. This comprise of test results for all the parameters tested, i.e. COD, BOD, TKN, NH_3-N, and fecal coliform; at compliance depth of the disposal fields, 1 foot below it, and at monitoring well locations. Results that were found to be below detection limit for any particular test have been indicated in these tables.

Average Concentrations

The first step taken in evaluating performance of these disposal systems was calculating average concentrations of the pollutants to get a preliminary assessment. For any particular monitoring location (either lysimeter or monitoring well) and parameter, average pollutant concentration was calculated by considering the entire set of results obtained. Each data set was then categorized into separate groups in terms of weather conditions as per criteria discussed earlier and average concentrations for warm, cold, wet, and dry seasons are computed for further comparison. Summary of average concentrations is given in Tables 5.1 through 6, respectively for 6 sites.

To look into the consistency of test results, corresponding standard deviations for the average values were found and listed in the Tables. Average results from lysimeter locations were compared to the discharge limits against which the adequacy of these disposal techniques was being assessed. While most of these average concentrations were found to be within the limits, the following could be indicated for further review,

- For the first subsurface drip irrigation sites (site 1), average value of BOD at compliance depth for warm season was found to be more than 30 mg/L; however there was only one instance when the data was obtained.
- In case of one of the sprinkler irrigation sites (site 3), average concentrations of COD at compliance depth was very high for all the seasons. This concentration was substantially reduced at 1 ft below this depth. It was also observed that the average TKN concentrations at compliance level were above 5 mg/L value for all the seasons, yet it was significantly reduced at higher depth. The average BOD value at compliance depth was found to be above the 30 mg/L limit for cold season.

The above findings indicated that further statistical analysis of the results needed to be performed for a more detailed investigation of functioning of these systems.

Table 5.1

Summary of Performance Evaluation Data for Drip Irrigation Site (Site No 1)

Parameter	Sampling Depth [c]	No of Data Points						Data				
		All[a]	Weather Condition					All[a]	Weather Condition			
			Wet	Dry	Warm	Cold			Wet	Dry	Warm	Cold
COD, mg/L	At compliance depth	24	[b]	[b]	9	15	Average	42.38	[b]	[b]	47.83	39.11
							Std. Dev.	23.28	[b]	[b]	13.27	27.56
	At 1 foot below compliance	24	[b]	[b]	12	12	Average	18.66	[b]	[b]	17.18	19.85
							Std. Dev.	11.17	[b]	[b]	9.57	12.50
BOD, mg/L	At compliance depth	6	[b]	[b]	1	5	Average	22.16	[b]	[b]	38.21	18.95
							Std. Dev.	22.15	[b]	[b]	[b]	23.15
	At 1 foot below compliance	11	[b]	[b]	5	6	Average	6.90	[b]	[b]	7.08	6.74
							Std. Dev.	3.62	[b]	[b]	3.74	3.86
TKN, mg/L	At compliance depth	19	[b]	[b]	8	11	Average	1.43	[b]	[b]	1.79	1.16
							Std. Dev.	1.20	[b]	[b]	1.63	0.82
	At 1 foot below compliance	25	[b]	[b]	13	12	Average	0.46	[b]	[b]	0.43	0.48
							Std. Dev.	0.23	[b]	[b]	0.22	0.27
NH_3-N, mg/L	At compliance depth	15	[b]	[b]	4	11	Average	0.40	[b]	[b]	1.02	0.18
							Std. Dev.	0.49	[b]	[b]	0.57	0.15
	At 1 foot below compliance	25	[b]	[b]	13	12	Average	0.06	[b]	[b]	0.09	0.02
							Std. Dev.	0.08	[b]	[b]	0.11	0.02
FC, cfu/100 ml [d]	At compliance depth	9	[b]	[b]	6	3	Average	2	[b]	[b]	3	0
							Std. Dev.	5	[b]	[b]	7	0
	At 1 foot below compliance	8	[b]	[b]	6	2	Average	18	[b]	[b]	20	14
							Std. Dev.	33	[b]	[b]	42	3

[a] All data considered, [b] Insufficient information, [c] Compliance depth is 4 ft, [d] Discharge limit – 30 mg/L BOD, 5 mg/L TKN, and 100 cfu/100 ml

Table 5.2

Summary of Performance Evaluation Data for Drip Irrigation Site (Site No 2)

Parameter	Sampling Depth [c]	No of Data Points						Data				
		All[a]	Weather Condition					All[a]	Weather Condition			
			Wet	Dry	Warm	Cold			Wet	Dry	Warm	Cold
COD, mg/L	At compliance depth	32	6	26	17	15	Average	89.10	54.73	97.04	81.46	97.76
							Std. Dev.	50.81	22.32	52.46	40.75	60.56
	At 1 foot below compliance	31	6	25	16	15	Average	93.69	55.83	102.78	97.97	89.13
							Std. Dev.	68.34	33.60	71.82	65.52	73.25
BOD, mg/L	At compliance depth	14	6	8	6	8	Average	8.73	7.73	9.47	10.04	7.75
							Std. Dev.	5.47	2.63	7.01	6.35	4.92
	At 1 foot below compliance	16	6	10	5	11	Average	11.29	12.61	10.50	13.78	10.16
							Std. Dev.	10.59	9.84	11.47	10.12	11.08
TKN, mg/L	At compliance depth	31	6	25	16	15	Average	2.14	1.07	2.39	2.02	2.26
							Std. Dev.	2.30	0.80	2.48	1.54	2.96
	At 1 foot below compliance	32	6	26	17	15	Average	1.73	0.99	1.92	1.88	1.55
							Std. Dev.	1.56	1.04	1.68	1.48	1.69
NH_3-N, mg/L	At compliance depth	31	6	25	16	15	Average	0.23	0.02	0.28	0.39	0.05
							Std. Dev.	0.48	0.01	0.53	0.64	0.06
	At 1 foot below compliance	29	6	23	14	15	Average	0.10	0.02	0.13	0.17	0.05
							Std. Dev.	0.28	0.02	0.31	0.39	0.07
FC, cfu/100 ml [d]	At compliance depth	14	4	10	6	8	Average	7	3	8	5	8
							Std. Dev.	14	4	16	6	18
	At 1 foot below compliance	17	5	12	5	12	Average	6	9	5	9	5
							Std. Dev.	10	14	9	14	9

[a] All data considered, [b] Insufficient information, [c] Compliance depth is 4 ft, [d] Discharge limit – 30 mg/L BOD, 5 mg/L TKN, and 100 cfu/100 ml

Table 5.3

Summary of Performance Evaluation Data for Sprinkler Irrigation Site (Site No 3)

Parameter	Sampling Depth [c]	No of Data Points						Data				
		All[a]	Weather Condition					All[a]	Weather Condition			
			Wet	Dry	Warm	Cold			Wet	Dry	Warm	Cold
COD, mg/L	At compliance depth	26	5	21	20	6	Average	377.10	260.71	404.81	345.18	483.49
							Std. Dev.	183.49	213.81	169.56	188.67	124.22
	At 1 foot below compliance	24	3	21	18	6	Average	51.35	83.17	46.80	44.81	70.96
							Std. Dev.	29.30	31.25	26.77	27.13	28.87
BOD, mg/L	At compliance depth	10	0	10	8	2	Average	19.76	[b]	19.76	17.13	30.19
							Std. Dev.	18.54	[b]	18.54	19.98	5.30
	At 1 foot below compliance	19	2	17	14	5	Average	5.07	3.24	5.28	5.13	4.89
							Std. Dev.	5.01	3.90	5.18	5.70	2.72
TKN, mg/L	At compliance depth	26	5	21	20	6	Average	9.71	9.17	9.84	9.71	9.74
							Std. Dev.	7.34	10.90	6.59	7.63	6.95
	At 1 foot below compliance	24	3	21	18	6	Average	0.66	0.72	0.65	0.64	0.72
							Std. Dev.	0.37	0.28	0.39	0.36	0.43
NH_3-N, mg/L	At compliance depth	27	6	21	19	8	Average	0.19	0.41	0.13	0.23	0.10
							Std. Dev.	0.39	0.77	0.17	0.45	0.12
	At 1 foot below compliance	25	4	21	18	7	Average	0.02	0.01	0.02	0.02	0.01
							Std. Dev.	0.03	0.02	0.03	0.03	0.02
FC, cfu/100 ml [d]	At compliance depth	10	0	10	9	1	Average	4	[b]	4	5	0
							Std. Dev.	7	[b]	7	7	[b]
	At 1 foot below compliance	20	0	20	17	3	Average	13	[b]	13	15	3
							Std. Dev.	28	[b]	28	30	5

[a] All data considered, [b] Insufficient information, [c] Compliance depth is 3 ft, [d] Discharge limit – 30 mg/L BOD, 5 mg/L TKN, and 100 cfu/100 ml

Table 5.4

Summary of Performance Evaluation Data for Sprinkler Irrigation Site (Site No 4)

Parameter	Sampling Depth [c]	No of Data Points						Data				
		All[a]	Weather Condition					All[a]	Weather Condition			
			Wet	Dry	Warm	Cold			Wet	Dry	Warm	Cold
COD, mg/L	At compliance depth	27	15	12	12	15	Average	58.00	57.78	58.27	50.66	63.87
							Std. Dev.	39.21	38.73	41.54	30.78	45.04
	At 1 foot below compliance	29	15	14	14	15	Average	52.42	46.28	59.01	31.51	71.94
							Std. Dev.	50.37	39.03	61.11	15.11	63.38
BOD, mg/L	At compliance depth	22	15	7	9	13	Average	11.58	11.05	12.71	8.45	13.75
							Std. Dev.	11.31	9.56	15.23	9.81	12.13
	At 1 foot below compliance	21	15	6	9	12	Average	8.76	10.81	3.64	7.20	9.94
							Std. Dev.	10.71	12.14	1.64	11.18	10.69
TKN, mg/L	At compliance depth	27	15	12	12	15	Average	1.17	1.00	1.39	1.16	1.19
							Std. Dev.	0.87	0.72	1.01	0.75	0.98
	At 1 foot below compliance	29	15	14	14	15	Average	1.31	0.94	1.72	0.78	1.81
							Std. Dev.	2.14	1.23	2.81	0.53	2.89
NH_3-N, mg/L	At compliance depth	26	15	11	12	14	Average	0.06	0.06	0.05	0.05	0.06
							Std. Dev.	0.07	0.08	0.04	0.05	0.08
	At 1 foot below compliance	27	15	12	11	16	Average	0.05	0.07	0.03	0.06	0.05
							Std. Dev.	0.07	0.09	0.04	0.05	0.09
FC, cfu/100 ml [d]	At compliance depth	19	14	5	8	11	Average	8	8	5	5	9
							Std. Dev.	9	8	11	2	11
	At 1 foot below compliance	18	12	6	9	9	Average	3	3	2	2	3
							Std. Dev.	4	5	3	2	6

[a] All data considered, [b] Insufficient information, [c] Compliance depth is 3 ft, [d] Discharge limit – 30 mg/L BOD, 5 mg/L TKN, and 100 cfu/100 ml

Table 5.5

Summary of Performance Evaluation Data for Mound (Site No 5)

Parameter	Sampling Depth [c]	No of Data Points						Data				
		All[a]	Weather Condition					All[a]	Weather Condition			
			Wet	Dry	Warm	Cold			Wet	Dry	Warm	Cold
COD, mg/L	At compliance depth	30	9	21	21	9	Average	12.97	11.84	13.45	12.07	15.04
							Std. Dev.	6.99	5.86	7.50	7.81	4.16
	At 1 foot below compliance	30	9	21	21	9	Average	11.73	9.38	12.74	11.95	11.22
							Std. Dev.	7.18	5.27	7.75	7.72	6.11
BOD, mg/L	At compliance depth	16	7	11	11	5	Average	2.99	3.46	2.64	2.75	3.53
							Std. Dev.	2.44	2.22	2.67	2.37	2.78
	At 1 foot below compliance	18	7	11	12	6	Average	3.06	2.84	3.19	3.15	2.87
							Std. Dev.	2.75	2.45	3.03	2.84	2.80
TKN, mg/L	At compliance depth	27	9	18	21	6	Average	1.20	0.44	1.52	1.44	0.65
							Std. Dev.	2.05	0.20	2.39	2.41	0.50
	At 1 foot below compliance	30	9	21	21	9	Average	0.81	0.40	0.98	1.00	0.36
							Std. Dev.	1.83	0.19	2.17	2.17	0.17
NH_3-N, mg/L	At compliance depth	30	9	21	21	9	Average	0.26	0.02	0.37	0.33	0.10
							Std. Dev.	0.83	0.03	0.98	0.99	0.11
	At 1 foot below compliance	30	9	21	21	9	Average	0.12	0.06	0.15	0.15	0.05
							Std. Dev.	0.24	0.06	0.28	0.28	0.03
FC, cfu/100 ml [d]	At compliance depth	15	9	6	12	3	Average	2	2	1	2	0
							Std. Dev.	4	4	3	4	0
	At 1 foot below compliance	20	9	11	15	5	Average	5	7	3	6	2
							Std. Dev.	9	12	6	11	2

[a] All data considered, [b] Insufficient information, [c] Compliance depth is 3 ft, [d] Discharge limit – 30 mg/L BOD, 5 mg/L TKN, and 100 cfu/100 ml

Table 5.6

Summary of Performance Evaluation Data for Mound (Site No 6)

Parameter	Sampling Depth [c]	No of Data Points						Data				
		All[a]	Weather Condition					All[a]	Weather Condition			
			Wet	Dry	Warm	Cold			Wet	Dry	Warm	Cold
COD, mg/L	At compliance depth	21	14	7	10	11	Average	92.88	91.19	96.28	92.04	93.65
							Std. Dev.	61.40	48.06	86.85	58.90	66.46
	At 1 foot below compliance	26	18	8	12	14	Average	39.36	40.08	37.73	48.97	31.38
							Std. Dev.	31.34	33.93	26.59	39.32	20.78
BOD, mg/L	At compliance depth	14	12	2	6	8	Average	5.77	6.41	1.95	7.03	4.83
							Std. Dev.	4.95	5.07	1.27	6.52	3.57
	At 1 foot below compliance	22	16	6	9	13	Average	5.60	6.52	3.15	6.40	5.05
							Std. Dev.	6.55	7.38	2.61	6.76	6.61
TKN, mg/L	At compliance depth	21	14	7	10	11	Average	2.82	2.97	2.51	3.76	1.96
							Std. Dev.	3.20	3.75	1.88	4.26	1.57
	At 1 foot below compliance	25	18	7	12	13	Average	0.72	0.68	0.84	0.85	0.61
							Std. Dev.	0.48	0.47	0.53	0.56	0.39
NH_3-N, mg/L	At compliance depth	21	14	7	10	11	Average	0.33	0.39	0.22	0.44	0.23
							Std. Dev.	0.31	0.35	0.18	0.37	0.21
	At 1 foot below compliance	26	18	8	12	14	Average	0.14	0.15	0.13	0.19	0.10
							Std. Dev.	0.12	0.12	0.12	0.12	0.11
FC, cfu/100 ml [d]	At compliance depth	13	11	2	6	7	Average	4	4	0	1	6
							Std. Dev.	7	8	0	3	9
	At 1 foot below compliance	23	18	5	11	12	Average	1	1	1	1	2
							Std. Dev.	2	2	2	2	2

[a] All data considered, [b] Insufficient information, [c] Compliance depth is 3 ft, [d] Discharge limit – 30 mg/L BOD, 5 mg/L TKN, and 100 cfu/100 ml

C_{95} Concentrations

Values of C_{95} concentration for contaminants were calculated for all the lysimeter locations in six sites. These are first calculated for full data set and then individually for separate groups categorized as warm, cold, wet, and dry seasons. Again, C_{95} values could be presumed to represent a maximum concentration one would expect with 95% confidence. The results obtained are given in Tables 5.7, 5.8, and 5.9, respectively for three types of systems. Some of these values for certain monitoring stations could not be computed as only a single observation was made. On comparing these C_{95} concentration values to the discharge limits, a meaningful and clearer picture of the functional aspect of different disposal techniques was achieved.

Subsurface Drip Irrigation System

In the first site (site 1), COD concentration was consistently varying over a short range for warm and cold seasons. The BOD concentrations were found to be higher than 30 mg/L at compliance depth and were reduced to about 10 mg/L or less at 1 foot below compliance (Table 5.7). Concentrations of other pollutants like TKN, NH_3-N, and fecal coliforms were found to meet the set limits at compliance depth. In absence of sufficient information on water levels at four monitoring wells for this site, collected data could not be categorized between wet and dry seasons. So, observations made on the results for this site had to be restricted to warm and dry seasons. The BOD, TKN, and fecal coliform levels in the second drip irrigation site were found to be within the stipulated limits at compliance depth for all weather conditions. It was noted that COD concentrations were high both at compliance depth and below for different seasons; but since, BOD concentration at this level was observed to be less, presence of relatively high COD values could be attributed to non-organic sources, such as chemicals, refractory materials, and a faction of COD possibly being contributed by oxygen demand exerted by reduced nitrogen species.

Sprinkler Irrigation System

High concentrations of COD and elevated levels of TKN were detected at compliance depth across all conditions in the first sprinkler system (site 3) and were substantially reduced at higher depths (Table 5.8). BOD concentrations also were found to be above the limit at compliance and were reduced appreciably below this depth. The ammonia and fecal coliform levels were within the prescribed limit. It would be relevant to mention that for this sprinkler system, the aerator of the ATU was found to be not operating and as such the aeration tank was essentially functioning as a septic tank. This explained the high amount of COD detected at compliance depth in the disposal field. However, these values were much more than that would be expected from partially treated wastewater. Part of this COD was contributed by inadequately treated wastewater from aeration unit and a fraction of it was exerted by the reduced forms of nitrogen species escaping stabilization from anaerobic conditions of the non-functional aeration unit. The balance portion came form materials already present in soil matrix and were of non-organic origin, as otherwise it would have showed up in increased level of BOD at compliance depth. Pollutant concentrations at other sprinkler site were found to be within limits at compliance depth for all weather conditions.

Table 5.7

C_{95} Concentrations for Drip Irrigation Sites

Parameter	Depth of Sampling	Site 1					Site 2				
		All[a]	Wet	Dry	Warm	Cold	All[a]	Wet	Dry	Warm	Cold
COD, mg/L	At Compliance depth	50.52	[b]	[b]	56.06	51.64	100.87	73.09	114.61	98.72	125.30
	1 foot below compliance	22.33	[b]	[b]	22.14	25.53	109.77	83.47	127.36	126.86	122.44
BOD, mg/L	At Compliance depth	40.36	[b]	[b]	a	41.02	11.32	9.89	14.17	15.26	11.05
	1 foot below compliance	8.88	[b]	[b]	10.65	9.92	15.93	20.71	17.15	23.43	16.21
TKN, mg/L	At Compliance depth	1.91	[b]	[b]	2.88	1.61	2.68	1.73	3.24	2.70	3.61
	1 foot below compliance	0.54	[b]	[b]	0.54	0.62	2.09	1.85	2.48	2.51	2.32
NH_3-N, mg/L	At Compliance depth	0.62	[b]	[b]	1.69	0.26	0.34	0.03	0.46	0.67	0.08
	1 foot below compliance	0.09	[b]	[b]	0.14	0.03	0.17	0.04	0.24	0.36	0.08
FC, cfu/100 ml[c]	At Compliance depth	5	[b]	[b]	8	0	13	7	18	10	20
	1 foot below compliance	41	[b]	[b]	54	27	10	23	9	23	10

[a] All data set is considered

[b] Data segregation could not be done

[c] Discharge limit – 30 mg/L BOD, 5 mg/L TKN, and 100 cfu/100 ml

Table 5.8

C95 Concentrations for Sprinkler Irrigation Sites

Parameter	Depth of Sampling	Site 3					Site 4				
		All[a]	Wet	Dry	Warm	Cold	All[a]	Wet	Dry	Warm	Cold
COD, mg/L	At Compliance depth	438.56	468.64	464.57	418.12	585.68	70.87	75.39	79.81	66.62	84.35
	1 foot below compliance	61.60	56.88	135.85	55.94	94.71	68.33	64.03	87.94	38.66	100.76
BOD, mg/L	At Compliance depth	30.51	30.51	[a]	30.54	53.85	15.73	15.40	23.90	14.53	19.75
	1 foot below compliance	7.06	7.47	20.65	7.83	7.48	12.79	16.33	4.99	14.13	15.48
TKN, mg/L	At Compliance depth	12.17	12.32	19.56	12.66	15.46	1.46	1.33	1.91	1.55	1.64
	1 foot below compliance	0.79	0.80	1.19	0.79	1.07	1.99	1.50	3.05	1.03	3.12
NH_3-N, mg/L	At Compliance depth	0.32	0.19	1.04	0.41	0.18	0.08	0.10	0.07	0.08	0.10
	1 foot below compliance	0.03	0.03	0.03	0.03	0.03	0.07	0.11	0.05	0.09	0.09
FC, cfu/100 ml [b]	At Compliance depth	9	8	[a]	9	[a]	11	12	15	6	15
	1 foot below compliance	24	24	[a]	28	12	6	6	4	3	7

[a] All data set is considered

[b] Discharge limit – 30 mg/L BOD, 5 mg/L TKN, and 100 cfu/100 ml

Mound System

The contaminant levels at compliance depth over the entire array of conditions studied in both the sites were within the discharge limits set for performance evaluation (Table 5.9). Some aspects of operational data for these two sites have been discussed later, when further data analysis was done.

Effect of Temperature on Treatment

Effect of warm and cold weather conditions on degree of treatment achieved in any particular disposal technique was analyzed by single factor ANOVA analysis for each of the five parameters. A summary of the results of the hypothesis testing is given in Table 5.10. Tests for a few cases could not be performed due to inadequate information. The ANOVA technique was used to test the hypothesis of equal mean concentrations of pollutants for warm and cold weather conditions at compliance depth at any particular site. Most of these tests failed to reject the corresponding hypothesis indicating that there was insufficient evidence to show any significant difference between the mean values of corresponding concentrations. This, in turn, signified that performance of individual disposal techniques at the 95% confidence level was unaffected by warm or cold weather conditions. Results indicated significant variation in average concentrations of NH_3-N for the first drip irrigation (site 1) and first sprinkler irrigation (site 3) sites.

Table 5.9

C_{95} Concentrations for Mound System Sites

Parameter	Depth of Sampling	Site 5					Site 6				
		All[a]	Wet	Dry	Warm	Cold	All[a]	Wet	Dry	Warm	Cold
COD, mg/L	At Compliance depth	15.14	15.47	16.27	15.01	17.62	115.99	113.94	160.06	132.17	129.96
	1 foot below compliance	13.96	12.65	15.66	14.86	15.01	49.86	54.00	55.55	42.15	41.22
BOD, mg/L	At Compliance depth	4.06	5.09	4.30	4.05	6.18	8.11	9.04	7.62	12.39	7.22
	1 foot below compliance	4.19	4.64	4.85	4.62	5.17	8.00	9.76	5.30	10.59	8.32
TKN, mg/L	At Compliance depth	1.84	0.56	2.42	2.35	0.96	4.02	4.75	3.89	6.23	2.82
	1 foot below compliance	1.38	0.52	1.80	1.82	0.47	0.88	0.87	1.23	1.14	0.80
NH_3-N, mg/L	At Compliance depth	0.52	0.04	0.74	0.70	0.17	0.45	0.56	0.35	0.66	0.35
	1 foot below compliance	0.19	0.10	0.26	0.26	0.07	0.18	0.20	0.21	0.25	0.15
FC, cfu/100 ml [b]	At Compliance depth	4	5	4	4	0	7	9	0	4	13
	1 foot below compliance	8	14	6	11	3	2	2	3	2	3

[a] All data set is considered

[b] Discharge limit – 30 mg/L BOD, 5 mg/L TKN, and 100 cfu/100 ml

Table 5.10

ANOVA Analysis for Effect of Weather Conditions on Extent of Treatment

Parameter (Dependent Variable)	Weather Conditions (Independent Variable)	Site 1	Site 2	Site 3	Site 4	Site 5	Site 6
		Drip Irrigation		Sprinkler Irrigation		Mound System	
		p-values of ANOVA compared against an α of 0.05					
COD, mg/L	Wet vs. dry	[a]	0.063	0.304	0.857	0.842	0.634
	Cold vs. warm	0.386	0.745	0.063	0.451	0.309	0.619
	Interaction	[b]	0.478	[b]	0.704	0.709	0.226
BOD, mg/L	Wet vs. dry	[a]	0.543	[a]	0.729	0.202	0.717
	Cold vs. warm	0.490	0.454	[c]	0.296	0.708	0.408
	Interaction	[b]	0.997	[a]	[b]	0.048	[b]
TKN, mg/L	Wet vs. dry	[a]	0.220	0.289	0.163	0.314	0.917
	Cold vs. warm	0.284	0.889	0.117	0.553	0.501	0.387
	Interaction	[b]	0.540	0.018	0.264	0.667	0.605
NH_3-N, mg/L	Wet vs. dry	[a]	0.231	0.000	0.699	0.445	0.578
	Cold vs. warm	0.000	0.326	0.000	0.829	0.661	0.166
	Interaction	[b]	0.322	0.000	0.931	0.691	0.531
FC, cfu/100 ml	Wet vs. dry	[a]	0.599	[b]	0.142	0.612	0.502
	Cold vs. warm	0.516	0.884	[c]	0.094	0.322	0.656
	Interaction	[b]	0.716	[b]	[b]	[b]	0.656

[a] - No data segregation could be done

[b] - No interaction could be worked out due to non-segregation of data

[c] - Insufficient data

Effect of Wet and Dry Weather on Treatment

A similar analysis was performed to look into the dependence of treatment by any given type of disposal field on wet or dry weather conditions. The technique followed was the same as followed previously and the results are summarized in Table 5.10. Contaminant concentrations were un-affected by the dry and wet weather conditions regardless of the disposal technique, excluding NH_3-N. Variation in NH_3-N concentrations in dry and wet conditions was observed for the first sprinkler irrigation site (site 3). As recorded in Table 5.4, average concentration of NH_3-N under wet conditions was more than that in dry conditions. Nitrification was affected during wet conditions as penetration of atmospheric air into soil was reduced. The conversion of NH_3-N into oxidized form of nitrogen was therefore decreased resulting in higher NH_3-N concentration.

Interactive Effect of Weather Conditions on Treatment

A probable interactive effect of weather conditions (such as cold-wet, cold-dry, warm-wet, and warm-dry) on the level of treatment provided by the disposal techniques was investigated across all the 6 sites (Table 5.10). Some analyses could not be performed due to insufficient data. Results showed that interactive weather conditions affected TKN and NH_3-N concentrations for the first sprinkler irrigation site (site 3) and BOD concentration for the first mound site (site 5). Variation observed in NH_3-N concentrations at site 1 due to individual weather conditions (e.g. dry-wet, and warm-cold) was believed to be the reason for their interactive effect also. Close review of the statistical analysis data could not pinpoint the combinations of weather conditions that resulted in variation of TKN concentrations for site 3 and BOD concentrations for site 5. Apart from these cases, the interactive effect did not adversely affect the operation of the disposal techniques indicating no statistically significant difference in the pollutant concentrations for various combinations of weather conditions.

Effect of Disposal Technique

Subsequent statistical analysis was done to quantify differences in treatment level achieved by different disposal technique types. The results are given in Table 5.11.

Table 5.11

ANOVA Analysis for Effect of Disposal Techniques on Treatment

Parameter (Dependent Variable)	Independent Variable			
	Individual Sites (all 6)	Drip vs. Sprinkler irrigation	Sprinkler irrigation vs. Mound	Drip irrigation vs. Mound sites
	p-values of ANOVA compared against an α value of 0.05			
COD, mg/L	0.000	0.000	0.000	0.021
BOD, mg/L	0.003	0.731	0.001	0.003
TKN, mg/L	0.000	0.001	0.001	0.996
NH_3-N, mg/L	0.174	0.050	0.102	0.953
FC, cfu/100 ml	0.250	0.545	0.049	0.387

Initially, this was done on a site-specific standpoint by comparing results collected from individual sites. The 6 sites, irrespective of the disposal technique installed, did not show any variation in NH_3-N and fecal coliform concentrations signifying effectiveness of all three techniques in reducing these pollutants to acceptable limits. All other parameter values showed variations.

Subsequently, data collected from a specific type of disposal technique was combined together to compare between any two types of disposal techniques. The COD concentrations showed significant difference when two types of disposal techniques were compared, similar to when results from individual sites were compared. Higher COD concentrations were found at one in each of the three types of disposal sites (i.e. sites 2, 3, and 6), as opposed to reason levels of COD in the other 3 sites. Reasons for such high COD levels were already discussed in previous sections. Variation in COD values as observed in the statistical analysis could be related to such difference in COD levels between two sites with similar disposal techniques. The BOD concentrations when compared between drip irrigation and sprinkler type system, and TKN concentrations when compared between drip irrigation and mound system, did not show significant variation. This could be interpreted as follows,

- Drip and sprinkler irrigation systems were consistently effective in reducing BOD concentrations from wastewater when compared to mound system.
- Sprinkler irrigation system might not prove to be as effective as the other two systems for reducing TKN concentration.

Though significant variations in concentrations of pollutants have been indicated by the statistical techniques used, most of the systems were found to either comply the set discharge limits or did indicate justified reasons for non-compliance. These observations did support the use of any of these disposal systems for effective on-site wastewater management.

Effect of Depth on Treatment

This analysis was performed in order to elaborate by statistical means any appreciable differences in extent of treatment received by wastewater as it percolated down through the disposal field; in other words to indicate any vertical variation in treatment. The primary objective of lysimeters being planted at two separate depths (at compliance and 1 foot below) in each disposal field was to investigate any such variation existed in extent of treatment. Such vertical difference in treatment was evaluated by performing ANOVA analysis on two sets of results collected at two separate depths for each of these disposal fields.

Initially, all data collected at compliance depth from any given type of disposal technique was combined and compared to a similar set of data at 1 foot below compliance to find significant variation amongst similar types of system (Table 5.12). Results indicated significant variation in concentrations of TKN and NH_3-N for drip irrigation, all parameters except fecal coliform for sprinkler irrigation, and COD and TKN for mound system sites. For drip irrigation site such differences in TKN and NH_3-N levels could be accounted for appreciable reduction of concentrations of these parameters from compliance depth to 1 foot below it specifically for site 1 (refer Table 5.1) that had affected the overall analysis.

Difference in overall performance of two sprinkler irrigation sites (malfunction of ATU at site 3) could be the reason for such variation in treatment levels at two depths. Significant reduction in COD and TKN concentrations at higher depth were noted for disposal field in site 6 (refer Table 5.6). Though this caused a statistically significant difference in concentrations at two depths, TKN concentration met the discharge limit at compliance depth and lower BOD values (Table 5.9) ensured acceptable level of treatment at this site.

Table 5.12

ANOVA Analysis for Effect of Depth on Treatment in Various Disposal Techniques

Parameter (Dependent Variable)	Sampling Depth (Independent Variable)	Type of Disposal System		
		Drip irrigation	Sprinkler irrigation	Mound system
		p-values of ANOVA compared against an α of 0.05		
COD, mg/L	Compliance depth vs. 1 foot below Compliance	0.325	0.000	0.011
BOD, mg/L		0.325	0.010	0.886
TKN, mg/L		0.033	0.000	0.009
NH_3-N, mg/L		0.007	0.022	0.088
FC, cfu/100 ml		0.317	0.662	0.876

To further look into the effect of depth on treatment, it was perceived that rather than doing a system-specific assessment, a site-specific analysis might prove to be a better approach. The output of such an analysis is summarized in Table 5.13. It was observed that there was no significant difference between the concentrations of pollutants at two depths for one in each type of disposal system; such as drip irrigation (site 2), sprinkler irrigation (site 4), and mound (site 5). As concluded earlier, disposal systems at these locations were also meeting the stipulated discharge limits at compliance depth.

Except for fecal coliform, other parameters in site 1 showed appreciable variation between the two depths. Amongst these other parameters, all but BOD were observed to be under the permitted limit at compliance. It was noted that reduction of BOD at 1 foot below compliance depth met the corresponding discharge limit which apparently suggested that addition of extra foot of depth to existing regulations might prove to be justified. However, a similar site with drip irrigation system showed acceptable performance data and met all the discharge norms satisfactorily.

Table 5.13

ANOVA Analysis for Effect of Depth on Treatment in Various Sites

Parameter (Dependent Variable)	Depth of Sampling (Independent Variable)	Site 1	Site 2	Site 3	Site 4	Site 5	Site 6
		Drip Irrigation		Sprinkler Irrigation		Mound System	
		p-values of ANOVA compared against an α of 0.05					
COD, mg/L	Compliance depth vs. 1 foot below Compliance	0.000	0.763	0.000	0.648	0.503	0.000
BOD, mg/L		0.037	0.423	0.003	0.407	0.946	0.935
TKN, mg/L		0.000	0.411	0.000	0.753	0.440	0.002
NH_3-N, mg/L		0.001	0.240	0.026	0.665	0.382	0.006
FC, cfu/100 ml		0.182	0.845	0.332	0.038	0.255	0.151

First drip irrigation site (site 1) had been dry over the sampling period and insufficient sample collection affected running tests for all the parameters. As a consequence, operating data collected for site 1 had not been found to be as exhaustive as that for site 2. Statistical techniques, by which these sites were being evaluated for performance, worked better and could be more conclusive with a reasonable amount of collected data. Hence, it could be inferred more conclusively that site 2 was performing acceptably well with a larger array of data set as its basis than concluding marginally inadequate functioning of site 1 having relatively less amount of data. So, despite having marginal variation in the operating data, it was concluded that drip irrigation system worked acceptably and addition of an extra foot to the existing compliance depth would not be necessary.

Significant variation in all parameters except fecal coliform at two depths for first sprinkler disposal field (site 3) had been found. Reason for higher values of BOD, TKN, and NH_3-N, recorded at compliance could be correlated to a non-functioning aerator unit at this site. The same argument was partially true for high concentrations of COD, as well. The filtering and adsorbing action of existing soil might be regarded as the reason for reduction of this extra amount of contaminants being disposed over the field. This in turn also caused the variation in concentration levels at two depths found in the analysis. For the second mound system (site 6) concentrations of COD, TKN, and NH_3-N at two different depths showed appreciable variation. However, the levels of TKN, and NH_3-N were found to be within discharge limits (Table 5.9). The same justification was true for variation in COD levels at compliance depth and 1 foot below it.

Site Soil Characteristics

The soil sample analysis for the six sites was done primarily to check if the disposal systems were installed in appropriate sub-surface conditions as stipulated by the regulations. Soil testing and analysis was done in line with the procedure mentioned in Chapter 3. Summary of such analysis and soil characterization is given in Appendix I.1 through 6, respectively for 6 sites. Supporting calculations and graphs for particle size distribution for each soil sample taken are furnished in Appendix J.1 through 24, respectively (i.e. 3 samples from each monitoring well location, 4 monitoring well locations at each site, and 6 sites, total 72 soil samples). It has been found that for drip irrigation sites subsurface soil condition varied from was medium or fine to loamy sand; for sprinkler irrigation sites it was sandy loam, loamy sand, and silt loam; for mound site it ranged from high to low plasticity clay, loamy sand, and sandy loam. Installation of drip and sprinkler irrigation systems over disposal field having predominantly sandy soil and mound system over field underlain by mostly clayey soil were in line with existing MDH regulations.

Groundwater Flow and Contaminant Movement

Groundwater was being used as a source of water supply in the project area. Hence, potential movement of pollutant from disposal field to groundwater was investigated to monitor any possible contamination.

Water levels recorded in monitoring wells over the sampling period had been furnished in Appendix K.1 though 6, respectively for all 6 sites. As per the criteria described earlier in the previous chapter, upstream and downstream monitoring wells were identified in each project site to ascertain the most probable direction of groundwater flow across the disposal field. The possible migration of pollutant from disposal fields was evaluated from two aspects,

- If the C_{95} concentration of any pollutant in the downstream monitoring well, as calculated from the data collected for that well, was noted to be more than the corresponding discharge limit, and
- If there was a significant difference between any pollutant levels as observed in upstream and downstream wells.

At first, concentrations of pollutants recorded over time in these identified monitoring wells were used to calculate their C_{95} concentrations (Table 5.14).

Table 5.14

C_{95} Concentrations for Upstream and Downstream Monitoring Wells

Parameter	Monitoring Well	Site 1	Site 2	Site 3	Site 4	Site 5	Site 6
		Drip Irrigation		Sprinkler Irrigation		Mound System	
COD, mg/L	Upstream	[a]	18.73	26.47	18.58	9.69	13.57
	Downstream	[a]	165.91	27.60	24.00	7.91	96.08
BOD, mg/L	Upstream	[a]	6.11	1.44	6.07	2.31	4.12
	Downstream	[a]	9.64	6.74	8.74	4.11	18.62
TKN, mg/L	Upstream	[a]	0.26	0.24	0.22	0.85	0.15
	Downstream	[a]	2.18	0.33	0.20	0.61	6.73
NH_3-N, mg/L	Upstream	[a]	0.02	0.11	0.04	0.04	0.02
	Downstream	[a]	0.05	0.08	0.04	0.02	1.06
FC, cfu/100 ml	Upstream	[a]	9	18	31	14	25
	Downstream	[a]	9	23	47	31	49

[a] – No definitive direction of groundwater flow could be established

The monitoring wells in site 1 were found to be dry over most of the sampling period and the data obtained from these wells were not sufficient enough to calculate or justify a probable groundwater flow pattern across the field. It was also understood that, as the wells were found to be dry in most instances, the groundwater table had been low in this particular site. This indicated that the direction of wastewater flow from the field was predominantly vertical instead of being lateral. So, it was not possible to perform the analysis to look into pollutant migration for the drip irrigation disposal field for this site.

From the values computed for C_{95} concentrations of various pollutants in downstream groundwater for both the sprinkler irrigation sites and first mound system site (site 5), it was observed that though there was change in pollutant levels in groundwater flowing across disposal fields, such concentrations did remain under the discharge limit. It was noted that there was significant rise in COD and TKN levels in groundwater in comparison to upstream concentrations in sites 2 and 6.

Corresponding C_{95} concentration values for BOD at compliance depths for these sites (refer Tables 5.7 and 5.9) were in compliance with discharge limit of 30 mg/L; hence, it could be concluded that such rise in COD levels were not related to organics of biological origin escaping treatment. Also, such levels of TKN in these two sites were shown to be complying with the discharge limit of 5 mg/L at compliance depth, precluding the possibility of increased TKN levels contributed by partially treated wastewater from disposal fields. ANOVA analysis was used to record any significant difference in concentrations of different parameters in upstream and downstream monitoring wells (Table 5.15).

In agreement with the C_{95} values calculated before, no significant difference in parameters values of upstream and downstream monitoring wells was seen for both sprinkler irrigation sites, and the first mound system (site 5). This, in conjunction with earlier argument, signified that the systems did not contribute any pollutant load to the groundwater flowing across these sites and values of parameters remained within the permissible range.

Table 5.15

ANOVA Analysis for Effect of Disposal Technique on Groundwater Quality

Parameter (Dependent Variable)	Direction of groundwater flow (Independent Variable)	Site 1	Site 2	Site 3	Site 4	Site 5	Site 6
		Drip Irrigation		Sprinkler Irrigation		Mound System	
		p-values of ANOVA analysis compared against an α of 0.05					
COD, mg/L	Samples from Upstream vs. Downstream Monitoring wells	[a]	0.000	0.504	0.265	0.118	0.000
BOD, mg/L		[a]	0.256	0.201	0.502	0.293	0.004
TKN, mg/L		[a]	0.000	0.237	0.641	0.602	0.067
NH_3-N, mg/L		[a]	0.041	0.819	1.000	0.355	0.000
FC, cfu/100 ml		[a]	0.818	0.658	0.227	0.457	0.448

[a] – No analysis could be performed

As indicated earlier, significant difference in COD and TKN levels were noted for upstream and downstream groundwater samples for disposal fields in sites 2 and 6. The reason for this was already identified. Though, results showed difference in NH_3-N concentrations of groundwater across disposal field in site 2, such values were found to be too low (refer Table 5.4 and 5.6) to affect its quality.

Summary of Analysis

From the above analysis, the three types of disposal techniques investigated were observed to be meeting the stipulated discharge limits (if operated adequately). No significant statistical variation in level of treatment achieved by these techniques were recorded which transpired that degree of treatment was independent of varying weather conditions and depth of sampling. Such findings also negated the possibility of revising the exiting regulations by addition of an extra foot to the present compliance depth. The study also provided information that no potential groundwater contamination could be detected as a result of transport of pollutants from the disposal fields. The level of nitrogen reduction in these fields was sufficient to bring down the TKN concentration below the prescribed limit (5 mg/L).

CHAPTER VI

ANALYSIS OF RESULTS: PROCESS MODIFICATION

This chapter will discuss and analyze results obtained from process simulations performed to look into the possibility of reducing nitrogen content of wastewater by suitably modifying the operation of any conventional activated sludge process. The information obtained will be incorporated, as appropriate, for possible application in the aerobic treatment units (ATUs) of on-site wastewater treatment systems. As mentioned earlier, two such proposals have been put forward for effective nitrogen removal through the typical nitrification and denitrification pathway; one by operating the aeration system at low dissolved oxygen (DO) concentration, and the other in intermittent aeration mode.

The process modeling of these two different biological processes is presented in the next two sections. The last section delineates the applicability of such process modifications for on-site treatment based on model predictions. It would be relevant to mention that, in assessing the extent of nitrogen removal that can be potentially forecast, the target nitrogen species were adopted as soluble ammonia-nitrogen (S_{NH}), soluble nitrate-nitrogen (S_{NO}), and soluble total nitrogen (S_{TN}). Extent of parallel reduction of organics from wastewater was noted by looking at the concentration of substrate or readily biodegradable COD (Ss). The basic idea of such an attempt was to see how best the operating parameters or philosophy of the activated sludge process can be manipulated to the advantage of achieving significant nitrogen reduction.

Simultaneous Nitrification-Denitrification Process

The first of the two processes was Simultaneous Nitrification-Denitrification (SND) where the two sequential biological processes were simulated in a typical activated sludge system. The initial set of simulations was aimed at identifying the optimum combination of operating parameter values for nitrification and denitrification to occur simultaneously in a completely mixed stirred-tank reactor (CSTR). Biological nitrification took place in presence of oxygen, and anoxic condition was required for subsequent denitrification. So, it was necessary first to identify appropriate operating conditions (DO and SRT) for sustenance of these two processes at the same time. Results obtained from such simulations had been presented in the form of four contour diagrams to better identify the operating conditions for SND process in terms of COD (S_S), S_{NH}, S_{NO}, and total nitrogen removal (S_{TN}) (Figure 6.1- a, b, c, and d), respectively. This figure had been developed from simulation results obtained for effluent concentrations of different nitrogen species with varying SRTs and operating DO levels. The corresponding graphs are provided in Appendices L.1 through 4.

Identification of Operating Conditions

Organic content of wastewater was predicted to be consumed almost entirely by adopting a 5-day SRT for DO level of ≥ 0.3 mg/L due to prolific heterotrophic growth. From the figures, it was indicated that effective total nitrogen (S_{TN}) removal was achieved for operating DO level of 0.4 mg/L in the aeration tank for a SRT of approximately 13 days. Beyond this point, TN removal almost remained unaffected with any further rise in SRT which was also seen for almost all higher DO levels. Though a better TN removal could be obtained with 0.3 mg/L DO and more than 20-day SRT, it was considered that adoption of higher SRT values could give rise to possible sludge bulking problem. Hence, higher SRT values were not considered for further analysis. Increased operating DO showed inhibition of denitrification process and higher effluent NO_3^--N concentration. Though higher DO levels required lesser SRTs for denitrification, overall TN removal suffered due to suppression of denitrification activity.

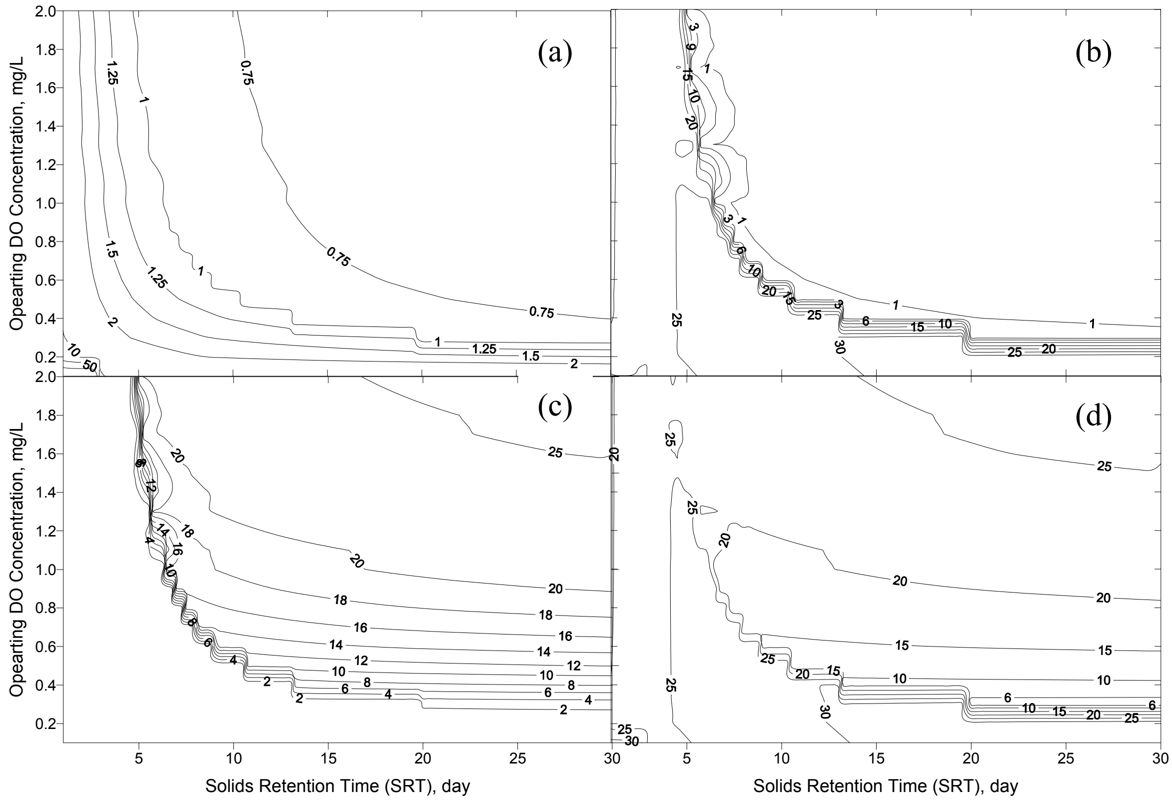


Figure 6.1 Effect of DO concentration and SRT on (a) Effluent Soluble COD (S_S), (b) Ammonia-Nitrogen (S_{NH}), (c) Nitrate (inorganic) Nitrogen (S_{NO}), and (d) Effluent Total Nitrogen (S_{TN}) in a SND system [a]

[a] Contour values indicate effluent concentrations and influent total nitrogen was 40 mg/L.

It was noted that a sudden drop in effluent COD with increase in SRT for any particular DO concentration could be correlated to a consequent reduction in effluent TN concentration for that corresponding DO level. This could be attributed to onset of heterotrophic denitrification process causing additional consumption of substrate. It was concluded that combination of 0.4 mg/L DO and 15-day SRT was optimal to render most efficient nitrogen removal by promoting SND process. Subsequent simulations had been performed adopting this particular combination of operating DO and SRT to study the process further.

Rittmann (2001) indicated that due to small maximum specific growth rate (μ_m) and large θ_x^{min} value, growth of nitrifiers is slow which implies sufficient time needs to be provided in an activated sludge system for their growth. The maximum specific autotrophic growth rate (μ_A) for aerobic nitrifiers in ASM1 model was expressed as (Henze et al., 1987a and b),

$$\mu_A = \hat{\mu}_A \left(\frac{S_O}{K_{O,A} + S_O} \right) \left(\frac{S_{NH}}{K_{NH} + S_{NH}} \right) X_{B,A} \qquad \text{Eqn 1}$$

For given influent S_{NH} concentration, and K_{NH} and $X_{B,A}$ values, the above form could be reduced to,

$$\mu_A = \hat{\mu}'_A \left(\frac{S_{NH}}{K_{NH} + S_{NH}} \right) X_{B,A} \qquad \text{Eqn 2}$$

Where, $\hat{\mu}'_A = \hat{\mu}_A \left(\frac{S_O}{K_{O,A} + S_O} \right)$ Eqn 3

The limiting minimum value of SRT or $[\theta_X]_{\min}^{\lim}$ was given by the following (Rittmann, 2001),

$$[\theta_X]_{\min}^{\lim} = \frac{1}{(\hat{\mu}'_A - b_A)} \qquad \text{Eqn 4}$$

Where, $\hat{\mu}'_A$ was a function of operating DO concentration (S_O). The value of $[\theta_X]_{\min}^{\lim}$ when plotted against operating DO concentration, Figure 6.2 was obtained,

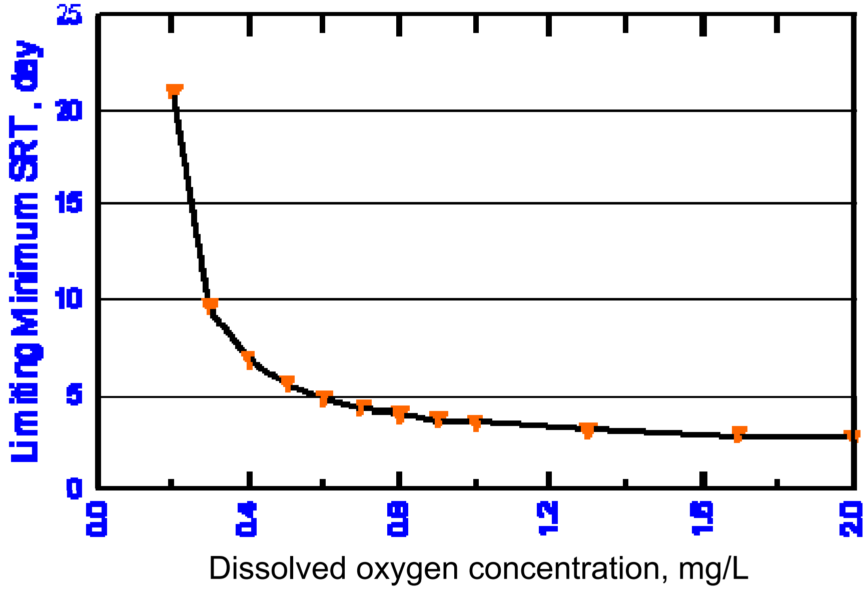


Figure 6.2 Limiting Minimum SRT at Various Operating DO for Autotrophic Growth of Nitrifiers

For the selected operating DO level of 0.4 mg/L, the value of $[\theta_X]_{\min}^{\lim}$ was calculated as 6.8 (app. ≈ 7) days using eqn 4 above. Also, the value of $[\theta_X]_{\min}^{\lim}$ for either aerobic or anoxic heterotrophs was much less than this computed for autotrophs. This signifies that the slow growth rate of autotrophic nitrifiers was predicted by the model and its growth was the limiting step in SND process. A value of 6.8 days for $[\theta_X]_{\min}^{\lim}$ thus justified the requirement of a 15-day SRT adopted (with about 2.2 safety factor) for occurrence of SND process in the reactor.

Other Factors Affecting SND Process

Once the operating parameters (DO concentration and SRT) required for SND process had been ascertained, the effect of other operating factors [BCOD:TKN ratio, sludge recycle ratio (R), hydraulic retention time (HRT)] were studied. Each of these parameters was varied in turn over a given range (Table 4.3) to observe its effect on various nitrogen species concentrations in the effluent and fractional nitrogen removal. The default values of these parameters were, BCOD:TKN ratio of 10, 12-hr HRT, and sludge recycle ratio (R) of 0.5.

Effect of BCOD:TKN Ratio

Simulations performed for lower BCOD:TKN ratios (e.g. 4.0, and 6.0) indicated that there was a significant drop in alkalinity in the reactor. This drop might be related to the fact that as proportion of nitrogen increased in the influent, more nitrogen (in NH_3-N form) was available for oxidation into NO_2^-/NO_3^- and only a portion of which eventually underwent denitrification. At the limiting operating DO level (0.4 mg/L), the results showed a gradual rise in effluent NO_3^--N concentration for increase in BCOD:TKN ratio from 5 to 7. Total nitrogen reduction for such BCOD:TKN ratios ranged from as low as 12% to 56%. The rise in NO_3^--N concentration might be the cause of rise in acidic condition inflicting a drop in alkalinity. Lower BCOD:TKN ratio in influent represented lesser amount of carbon source available in comparison to that with higher BCOD:TKN ratios for the same amount of nitrogen present in wastewater. As a result, for lower ratios, denitrification was predicted to be suppressed due to inadequate carbon source (i.e. heterotrophs consuming most of the substrate) despite other conditions being favorable (e.g. sufficiently low DO, adequate SRT, sufficient NO_3^--N concentration).

Nitrogen removal increased almost linearly under the given operating conditions as influent BCOD:TKN ratio was increased from 7 to 12. Beyond this point, fractional nitrogen reduction was unaffected by any further increase in this ratio. As noted from the results, beyond a BCOD:TKN ratio of 10, more than 75% total nitrogen removal could be achieved under the conditions studied. This might be due to the fact that a balance was struck at this range, and conversion of NH_3-N to NO_3^--N by oxidation and availability of carbon source for effective denitrification had reached optimum levels. Over 90% nitrogen removal was predicted for BCOD:TKN ratios of 12 and above (Figure 6.3). The results also indicated that BCOD:TKN ratio of influent wastewater was an important determinant of the extent of nitrogen removal that could be achieved by this process.

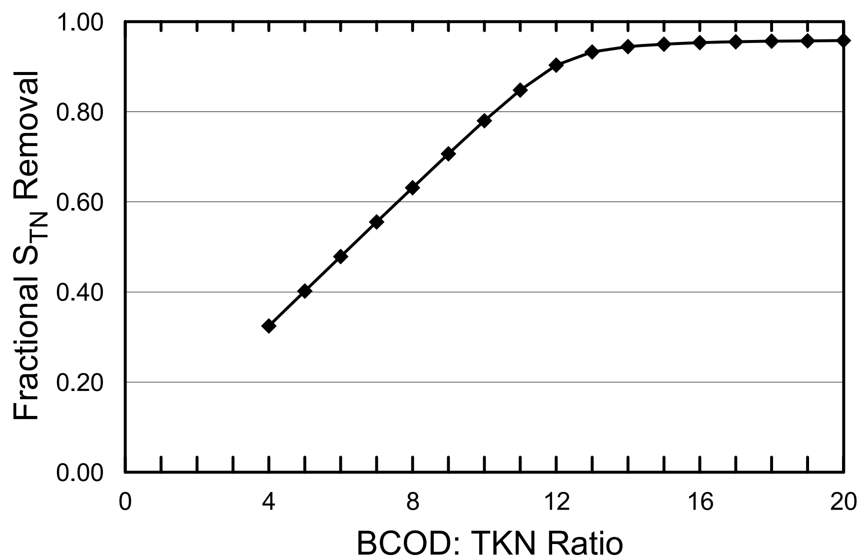


Figure 6.3 Effect of Influent BCOD: TKN Ratio on Nitrogen Removal

Effect of Hydraulic Retention Time (HRT)

Nitrogen removal when plotted against HRT showed a sudden rise in fractional nitrogen removal from 22% for 5-hr to about 80% for 7-hr HRT. Beyond a HRT of 8 hr, nitrogen removal remained unaffected by any further increase in HRT (Figure 6.4). Denitrification suffered for a 4-hr HRT and below as no oxidation of NH_3-N to NO_3^--N seemed to have occurred which was a precondition for nitrogen removal. Reduction in NH_3-N and COD was also noted to increase significantly as the HRT was increased to 8 hr or more signifying onset of denitrification. This indicated that under the given conditions, there was some minimum retention time that had to be provided. Once this HRT was provided, conditions become favorable for denitrification and significant nitrogen reduction occurred. In fact, this 8-hr HRT represented a "threshold" value beyond which SND process remained virtually unchanged by any further increase in HRT, yet for HRTs of 6 hr or less, denitrification seemed to have been suppressed.

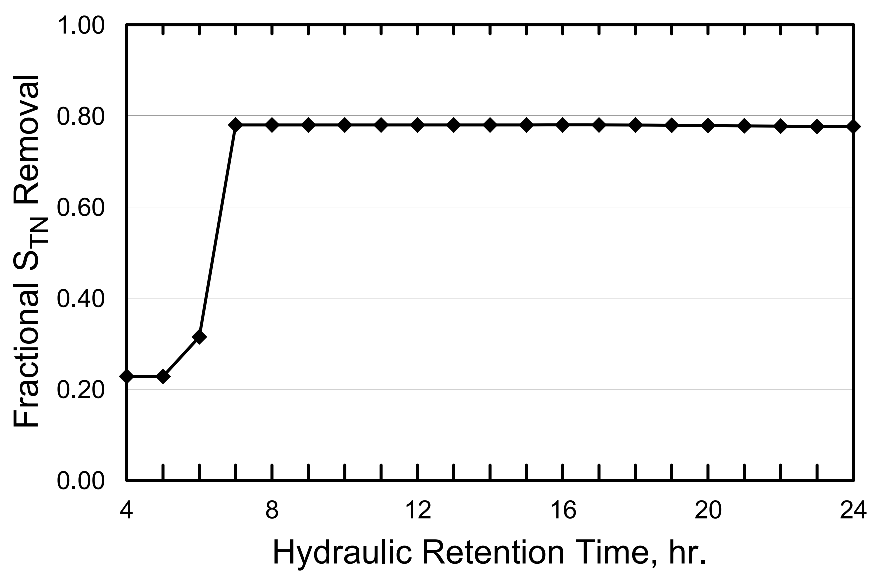


Figure 6.4 Effect of Hydraulic Retention Time (HRT) on Nitrogen Removal

Effect of Sludge Recycle Ratio (R)

Sludge recycle ratio (R) did not have any significant effect on the process and nitrogen removal was unaffected under the operating conditions. The 15-day SRT maintained was predicted to be adequate for occurrence of SND process and under the CSTR flow regime, even a very low amount of recycling (e.g. 0.25) proved to be sufficient enough to keep the process running.

Sensitivity Analysis of SND Process

Stochastic simulation were performed adopting the influent characteristics of wastewater and other operating parameter values as 0.4 mg/L DO, 15-day SRT, 12-hr HRT, and 0.5 recycle ratio (R). A total no of 1,000 Monte Carlo simulations were done with different values of 14 model parameters (Y_H, b_H, μ_H, K_S, K_{NO}, $K_{O,H}$, Y_A, μ_A, b_A, K_{NH}, $K_{O,A}$, k_h, K_X, η_h), selected as per log-normal probability density fraction (PDF) as described by Cox (2004). The PDFs of steady state effluent concentration of total nitrogen (S_{TN}) was plotted (Figure 6.5). Results were analyzed to examine the sensitivity of SND performance on selected dependent variables (e.g. S_S, S_{NH}, S_{NO}, S_{TN}, etc.), and sensitivity of overall nitrogen removal in order to assess its reliability from the standpoint of these parameter values. In Figure 6.5, stochastic simulation results predicted a process reliability of 22% which was the cumulative probability of the process meeting the results as predicted by discrete simulation.

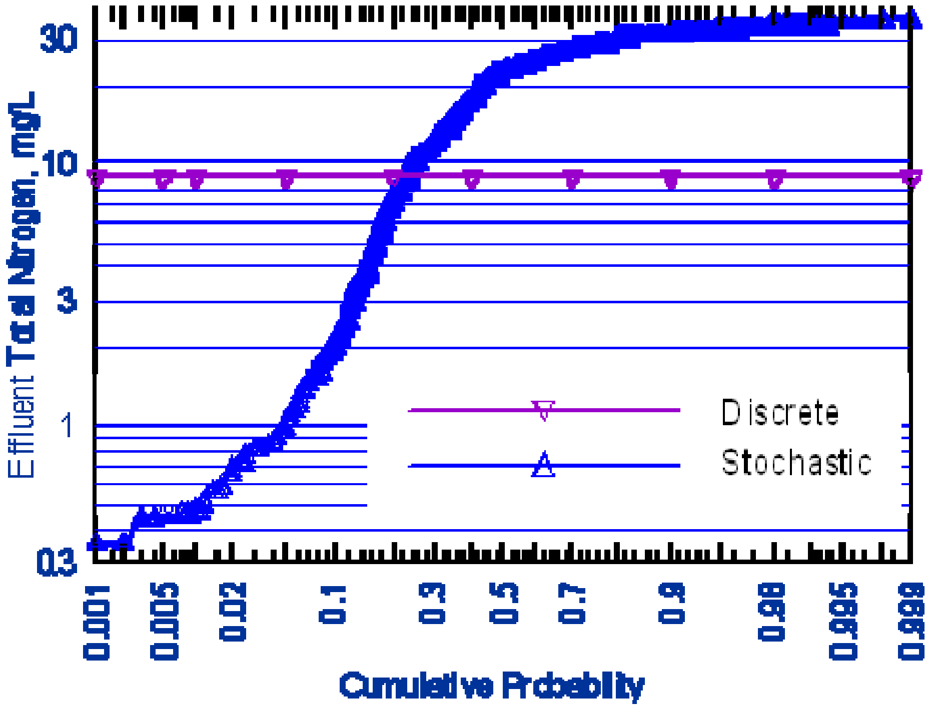


Figure 6.5 Stochastic Simulation Results of Effluent Total Nitrogen (S_{TN}) in SND system

A sensitivity analysis was performed by calculating the Spearman rank correlation matrix using SAS on the results of these simulations to identify those parameters (out of the 15) which effectively influenced the SND process. The output of such an analysis on different (S_{NH}, S_{NO}, and TN) types of effluent parameters is given in Appendix M.1 and 2. Strongest positive correlation was shown by oxygen half reaction constant for autotrophs ($K_{O,A}$) and most significant negative correlation was indicated by maximum specific autotrophic growth rate (μ_A) on overall nitrogen removal.

Certain observations made from this sensitivity analyses were,

- Since the resulting safety factor for growth of nitrifiers was low (~ 2.2), their growth was sensitive to variation in process parameters values that in turn adversely affected the SND process

- While the structure of ASM1 was suitable for modeling SND process, specific model parameters might have had to be calibrated for SND to more accurately model and simulate the process
- The PDFs defining the variability of different process parameters was considered to be separate, but some of these might have joint or connected PDFs.

Summary and Comparison with Earlier Studies

The model predictions pointed out some critical aspects of the process; (a) it was indicated that critical operating conditions to promote SND process were identified as 0.4 mg/L of DO in aeration tank and 15-day SRT, (b) TN (S_{TN}) removal was dependent on influent BCOD:TKN ratio, and (c) effect of recycle ratio (R) and HRT on nitrogen removal was not appreciable beyond certain min values of 0.25, and 6 hr, respectively. More than 75% of TN removal was predicted by the model for adopting optimum values of these operating parameters identified as 0.4 mg/L DO, 15-day SRT, 12-hr HRT, and 0.5 sludge recycle ratio for the influent wastewater having a BCOD:TKN ratio of 10.

Available literature indicated that in an experimental study, Elisabeth et al. (1996) reported that at an optimum DO of 0.5 mg/L (with 9.4 TCOD:TKN ratio, 18-hr HRT, and 15-day SRT), the two reaction rates (for nitrification and denitrification) would be similar and this might lead to complete SND. Another lab scale study conducted by Zeng et al. (2003) achieved < 1 mg/L effluent TN at 0.5 mg/L DO concentration and 15-day SRT. Significant nitrogen removal was also reported by Bertanza (1997) in pilot and real scale scenario at 0.3 to 0.5 mg/L operating DO concentration. However, literature is limited on recognizing the process parameters whose variability might significantly affect the occurrence and performance of SND process from total nitrogen reduction point of view.

Simulations performed in the WEST simulation platform (Vanhooren et al., 2003) by Insel (2006) predicted improved TN removal at 0.4 mg/L DO concentration with 20-day SRT and an influent BCOD:TKN ratio of 8. It was identified that at a DO set point of $K_{O,A} \geq S_{Oset} > K_{O,H}$ ensures high nitrogen removal with default stoichiometric and kinetic parameters. A study conducted by Lukasse et al. (1998) indicated that in ASM Model no 1, both SND and temporally separated nitrification and denitrification process might be optimal at limiting DO keeping in view the uncertainly associated with oxygen half reaction constants of autotrophic ($K_{O,A}$)and heterotrophic ($K_{O,H}$) biomass. In the simulations performed under the present study strong positive correlation was shown by oxygen half reaction constant for autotrophs ($K_{O,A}$) and significant negative correlation by maximum specific autotrophic growth rate (μ_A) on overall nitrogen removal. The forecasts of this study was in close agreement to the suggestion put forward by Münch et al. (1996) and Insel et al. (2005) that nitrification and denitrification could be achieved simultaneously at a reduced DO level of about 0.5 mg/L.

Intermittent Aeration System

Another modification of aeration system for nitrogen removal was investigated by operating the aeration system intermittently in an activated sludge process (ASP). The analysis presented here was first done with a surface aeration type system and later repeated for diffused aeration. Some initial simulations were run to identify the operating conditions for effective nitrogen reduction by an intermittently aerated activated sludge process. In these simulations, it was indicated that power input (in kW) to the reactor affected nitrogen removal over different anoxic time fractions in an intermittent aeration type ASP system with specified SRT and cycle time. The graphical output of such simulations is furnished in Appendices N.1 and N.2.

After having run these initial set of simulations for identifying critical operating parameters for the process, the system was modeled as a conventional activated sludge process with 2 mg/L DO concentration to compute the oxygen uptake rate of biomass (Table 4.5) for different SRT values. These uptake rates were then used to find the oxygen requirement of biomass which ultimately translated into required power input for the intermittent aeration system to run at the same DO level (during the aerobic cycle) at the corresponding SRT. These power inputs were adopted to run subsequent simulations (e.g. 24.71 kW for 15-day SRT). Interpretation of results and information obtained from the exercise are discussed below.

Nitrogen Removal

Results obtained from the simulations are presented in the form of four contour diagrams for effluent concentrations of soluble ammonia-nitrogen (S_{NH}), soluble nitrate-nitrogen (S_{NO}), soluble total nitrogen (S_{TN}), and readily biodegradable COD (S_S) concentrations (Graphs 6.6-a, b, c, and d), respectively, to identify the combined effect of cycle time and anoxic time fraction on the process at 15-day SRT. It was predicted that an increase in anoxic fraction beyond 0.5 (or 50%) adversely affected nitrification resulting in accumulation of S_{NH} and a consequent drop in effluent S_{NO} irrespective of the cycle time chosen (Fig 6.6-a and b).

In turn, denitrification is affected by insufficient concentration of the electron acceptor (S_{NO}). Conversely, for the anoxic phase, reduction in its time fraction (below

30%) depicted predominant nitrification and increase in effluent S_{NO}. As this fraction decreased, the available anoxic period became shorter resulting in suppression of denitrification and a drop in overall nitrogen reduction. It was shown (Fig 6.6-d) that effluent COD concentration increased at higher anoxic fractions (0.7 to 0.9) as a result of reduced denitrification occurring due to inadequate conversion of S_{NO}. A combined scenario (Fig 6.6-c) indicated optimum nitrogen removal between 0.4 and 0.5 anoxic time fraction and 2 and 4-hr cycle time.

Utilization of S_{NH} and Depletion of S_{NO}

In aerobic phase, ammonia nitrogen (S_{NH}) was converted into nitrate nitrogen (S_{NO}) by autotrophic nitrification to an extent when S_{NH} concentration either approached or droped below the K_{NH} value, i.e. ammonia half saturation constant for nitrifiers. After this, nitrification was found to have been subsided. For longer cycles ($\geq$ 6 hr) with extended aerobic period, such S_{NH} concentration was reached in about three quarters of the time into aerobic phase (Fig 6.8-b).

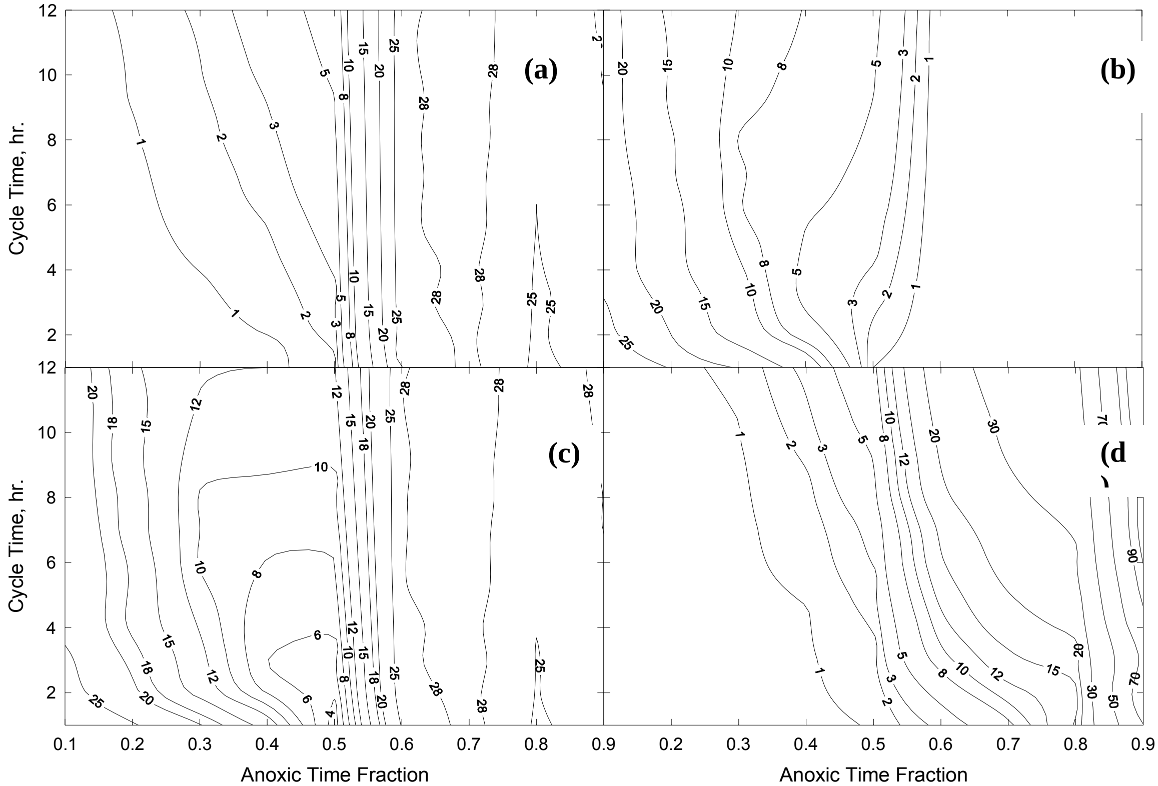


Figure 6.6 Effect of Cycle Time (CT) and Anoxic Time Fraction (AF) on (a) Effluent Ammonia-nitrogen (S_{NH}), (b) Nitrate (inorganic) nitrogen (S_{NO}), (c) Total nitrogen (S_{TN}), and (d) Soluble COD (S_S) in Intermittently Aerated System [a]

[a] Contours represent effluent concentrations, influent concentrations given in Table 4.2

Beyond this point, there was too little S_{NH} available in the reactor and influent could not sustain nitrification at previously observed rate. This indicated that for longer aerobic phases, some fraction of aeration time was not effectively utilized. On the other hand, the aerobic time phase was appropriately utilized for optimum nitrification of S_{NH} in case of shorter 2 hr to 3-hr cycles (Fig 6.7-b and 6.8-a). For such cycles, the effluent S_{NH} concentration approached the K_{NH} value closer to the completion of aerobic phase, causing better nitrogen reduction. Also, for a very short cycle (1 hr), the time available for aerobic phase might have been insufficient for establishing nitrification (Fig 6.7-a). Therefore, the results indicated that intermittent aeration system was more effective for shorter cycle times irrespective of the SRT range (15 – 25 day) studied.

In anoxic phase, it was indicated that for very long cycle time (12 hr) S_{NO} available from aerobic phase was depleted almost half way through the anoxic phase causing a shortage of electron acceptor for denitrification, negatively impacting nitrogen removal. The utilization of S_{NO} was observed to be coinciding with the completion of anoxic phase for shorted cycles of 2-hr and 3-hr (Fig 6.7-b and 6.8-a, respectively). This premature depletion of NO_3^- -N for longer cycles affected of nitrogen reduction by denitrification and overall nitrogen removal suffered. But for shorter cycles, S_{NO} remained available over the entire anoxic phase (Fig 6.7-b and 6.8-a) and denitrification was sustained. For very short cycle time (1 hr) accumulation of S_{NO} was noted probably due to the short time available for the onset of denitrification (Fig 6.7-a).

Heterotrophic reduction of organic COD was caused by denitrification under anoxic phase. It was indicated that in the anoxic phase of longer cycles, denitrification quite logically started to subside as soon as available S_{NO} became exhausted and this process, being the only probable pathway of organic COD consumption in anoxic phase, could be correlated to a consequent rise in effluent S_{COD} (Fig 6.8-b). So, with very long cycle time, not only denitrification suffered, it also resulted in increased effluent COD, concentration of which could rise to appreciable level. In contrast, for shorter cycle time, available COD would be consumed by denitrification (Fig 6.7-b and 6.8-a).

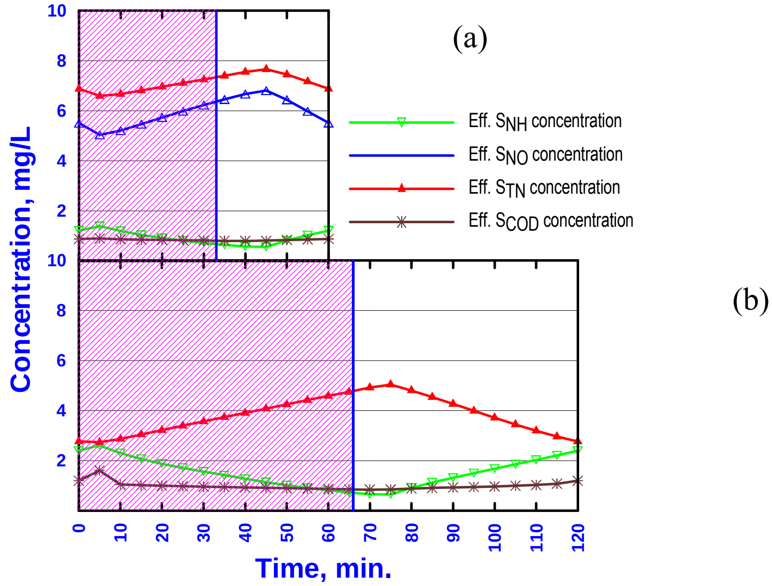


Figure 6.7 Profile of Effluent Ammonia-Nitrogen (S_{NH}), Nitrate (inorganic) Nitrogen (S_{NO}), total nitrogen (S_{TN}), and soluble COD (S_S) over (a) 1-hr and (b) 2-hr Cycle Times [a]

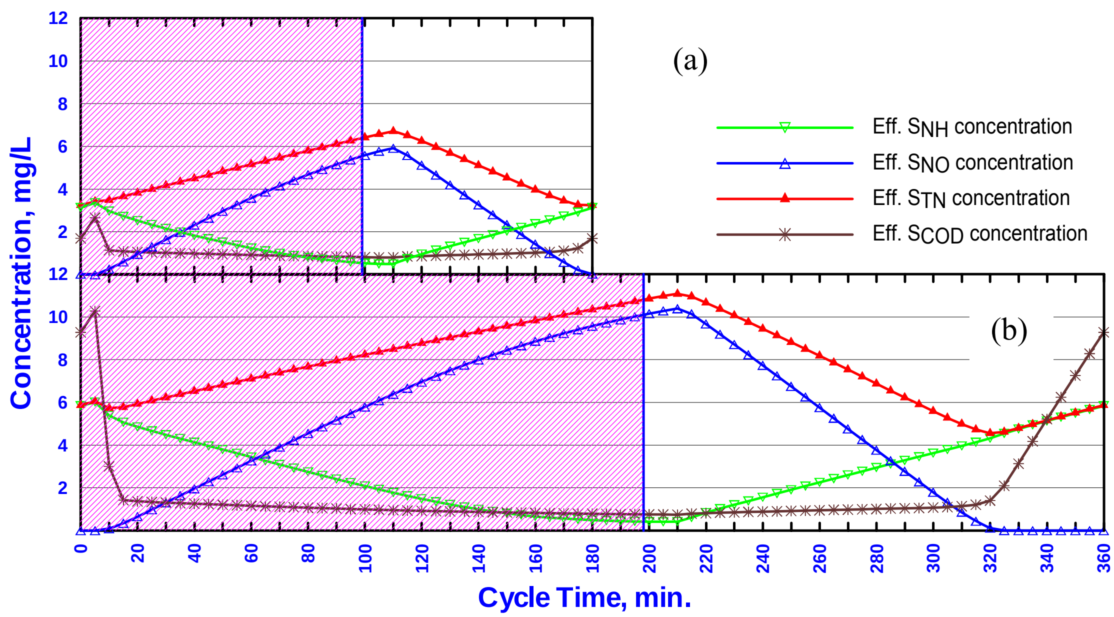


Figure 6.8 Profile of Effluent Ammonia-Nitrogen (S_{NH}), Nitrate (inorganic) Nitrogen (S_{NO}), total nitrogen (S_{TN}), and soluble COD (S_S) over (a) 3-hr and (b) 6-hr Cycle Times [a]

[a] The shaded portion represents aerobic phase and influent total nitrogen was 40 mg/L

Effect of total cycle time (CT) on nitrogen removal for specified operating conditions is given in Appendices O.1 and O.2 that indicated shorter cycle times (e.g. 2 to 3-hr) was effective in significant nitrogen reduction.

Anoxic Time Fraction and Operating DO Profile

As the anoxic time fraction was varied for adopted cycle time (3 hr), it was predicted that there was a range of anoxic time fraction over which maximum nitrogen removal occurred for any given SRT (15-day). This range was observed to widen slightly for higher SRT values (for 20 and 25-day) with other operating factors remaining constant, probably indicating a marginally wider range of anoxic time fraction for optimum process performance (Fig 6.9-a).

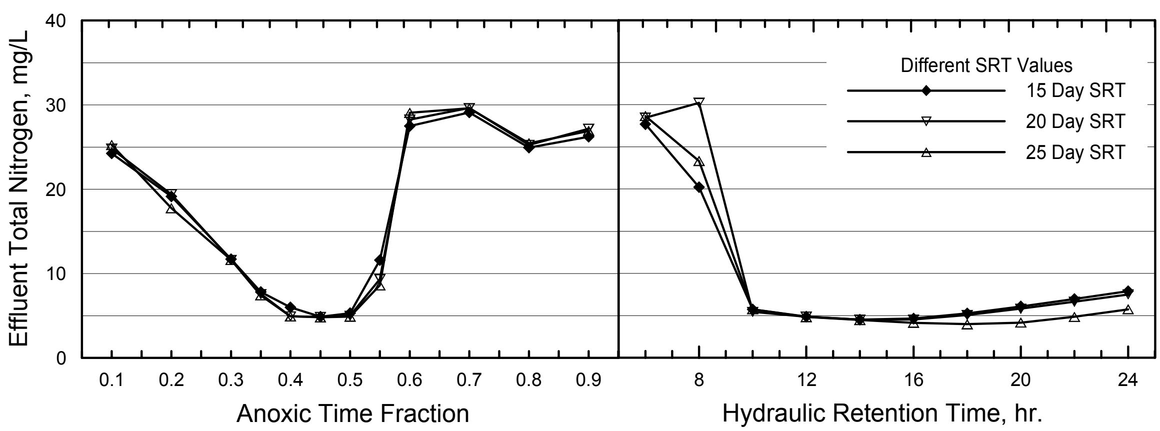


Figure 6.9 Effluent Total Nitrogen (S_{TN}) Concentration against Varying (a) Anoxic Time Fraction and (b) HRT for different SRT values adopted

Generally, it was noted that once the aerobic phase started and air supply was resumed in the reactor, DO concentration increased sharply in the initial phase.

Subsequently, this high rate of rise in DO subsided but DO concentration continued to increase and reached 3 to 5 mg/L depending on the expanse of aerobic phase. Once aeration was stopped, DO depleted quickly and once it had been reduced enough to ensure anoxic condition, denitrification was triggered and effluent S_{NO} started to decline. However, in case of very short cycle time (1-hr), it seemed that both the initial lag period for DO to reach a concentration high enough to start nitrification and the time taken for DO to deplete to a concentration low enough to prompt denitrification, became comparable to the duration of aerobic and anoxic time phases, respectively (Fig 6.10-a). This reduced the effective time segments over which actual nitrification and denitrification could occur. Such rising and falling of DO level in the reactor was found to be optimally balanced for shorter cycles when compared to that for longer cycles (Fig 6.11-a and b).

Hydraulic Retention Time (HRT) and Recycle Ratio

For a specific cycle time (3 hr) and anoxic time fraction of (0.45), simulation results predicted that for any HRT of 10-hr or higher, reasonably high level of nitrogen reduction could be achieved for all the three SRTs adopted. A marginal decrease in nitrogen reduction was noted when HRT was increased beyond 16-hr for 15 and 20-day SRT (Fig 6.9-b). No apparent effect could be found on nitrogen-removing potential of this process over the range of recycle ratios studied (0.5 to 3.0). A recycle ratio of 0.5 was found to be sufficient for effective nitrogen reduction.

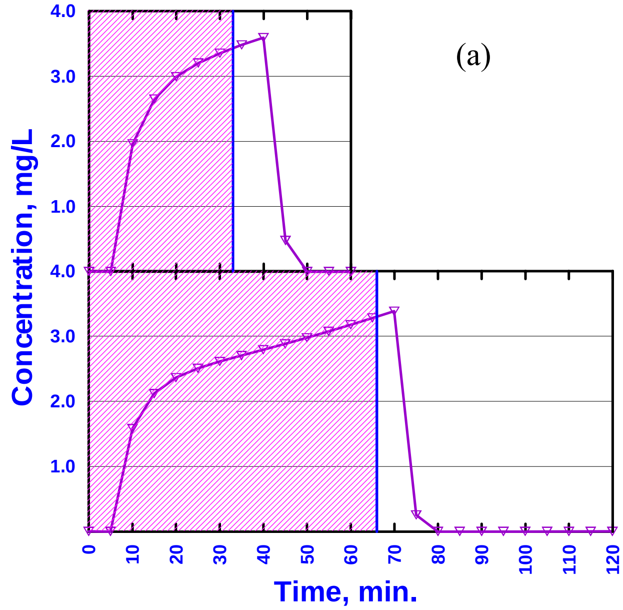


Figure 6.10 Operating DO concentration profile over complete (a) 1-hr and (b) 2-hr cycle times [a]

[a] The shaded portion represents aerobic phase

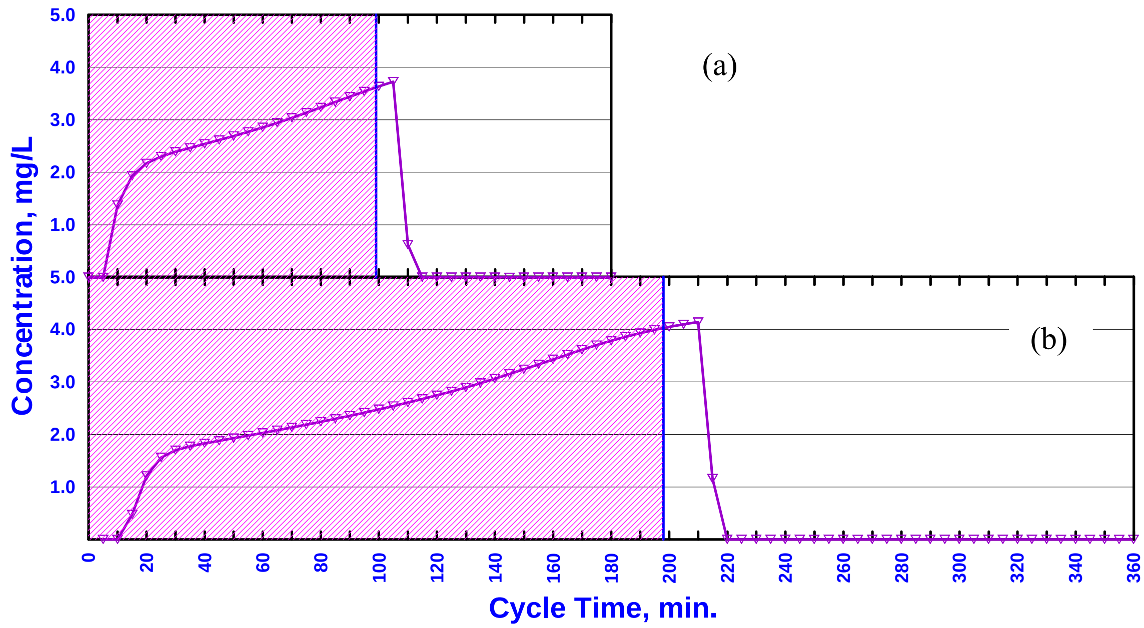


Figure 6.11 Operating DO concentration profile over complete (a) 3-hr and (b) 6-hr cycle times [a]

[a] The shaded portion represents aerobic phase

Other Operational Aspects

Selecting the optimized values of operating parameters as 3-hr cycle time, 0.45 anoxic time fraction, and 12-hr HRT, increased power input caused slight decrease in nitrogen removal irrespective of the SRT adopted. So, as per the model forecast, the power input taken for previous set of simulations (24.71 kW or about 50 kW/10^3 m^3) was found to be most efficient.

A similar analysis was performed for diffused aeration system where aeration was controlled by adjusting volumetric air flow into the reactor (m^3/min/10^3 m^3). In line with the results predicted for surface aeration, simulation output indicated that maximum nitrogen removal could be attained within a cycle time of 2 to 3-hr, anoxic time fraction of 0.4 to 0.5, and effect of recycle ratio on process performance was minimal. The various aspects discussed earlier, like exhaustion of S_{NH} in aerobic phase, depletion of S_{NO} and substrate in anoxic phase, too, were found to hold true for such a system, too. However, nitrogen removal was found to increase marginally with rise in HRT beyond 8-hr as opposed to that indicated in surface aeration system where a range of HRT was found to be most effective. Optimum air flow rate is found to be 16.48 m^3/min (i.e. app. 33 m^3/min/10^3 m^3).

Comparison with Previous Study Results

The model predictions seemed to be in limited agreement with other research findings keeping in view the difference in operating conditions and other factors considered. Hanhan et al. (2002) conducted a modeling study to retrofit an activated sludge system to intermittent aeration mode with 14-day SRT, 12-hr HRT, and an influent with COD:N ratio of 8.3. The system performance was reported to be most effective at 0.5 AF (anoxic fraction) and 75% average total nitrogen removal was achieved. It was concluded that selection of a CTR (cycle time ratio, i.e. ratio of cycle time and HRT) of 0.25 (i.e. 3-hr cycle time) seemed to have established the necessary conditions for nitrogen removal. Though the cycle time and anoxic time faction identified were within the respective ranges as found in this study, the aspect of optimizing cycle time and HRT by recognizing their individual effect on the process was dealt separately here. After a calibration study of wastewater treatment plant having aerobic, anoxic, and intermittently aerated tanks in series with ASM1, Lessard et al. (2007) concluded that sequences of aerated/non-aerated periods could be optimized for high level of nitrogen removal and that the impact of respective electron acceptor could also be evaluated, which were in line with the results obtained in the present study.

After conducting a full-scale study with biological reactor, Habermeyer and Sánchez (2005) concluded that aeration control was a critical parameter to simultaneously reduce COD and nitrogen and DO concentration had no significant effect on nitrogen removal. Their study results showed that longer on-off aeration time (i.e. 180 min on and 160 min off), in contrast to what was predicted here, and 1.5 – 1.7 mg/L DO set point could achieve about 70% and 90% of COD and TKN removal, respectively, without any accumulation of either NO_2^--N and NO_3^--N. Hidaka et al. (2002) reported high level of nitrogen removal with two continuously fed tanks in series and operated in intermittent aeration mode. The cycle time adopted was 90 min for each of these tanks with AF varying as 0.72 – 0.67 and 0.67 – 0.5 in the two tanks, respectively. The other operating parameters were taken as 30-hr HRT, 100 – 200% return sludge ratio, 10 to 25-

day SRT, and 0.6 – 1.0 mg/L DO set point. Though the cycle time and AF somewhat agreed to the range predicted in this modeling study, variation in results may be attributed to difference in other operating conditions considered. In a most recent study, total inorganic nitrogen (TIN) removal below 10 mg/L was predicted for a range of anoxic time fraction from 0.25 to 0.58 (Choubert et al., 2009a). This study also indicated the requirement of use of updated ASM1 model parameters for improved prediction of nitrogen removal, and these parameters were identified as, yield factor for heterotrophic biomass (Y_H), maximum specific growth rate for autotroph ($\mu_{m, A}$), and autotrophic biomass decay rate (b_A). It was confirmed (Choubert, 2009b) that plants were run at an average cycle time between 2 to 3-hr with 8 to 12 cycles per day, and 15-day SRT. The cycle time was taken as 2-hr during daily operation with adequate NH_4^+-N and COD present, and 3-hr during the night when these loadings dropped.

Inferences

Simulation of biological nitrogen removal by intermittent aeration type activated sludge process predicted to achieve effective and consistent nitrogen reduction. It was found that under the conditions stated, there existed a number of operating factors which were required to be adjusted for efficient process performance. The factors were identified as total cycle time (CT), anoxic time faction (AF), and hydraulic retention time (HRT), and it was indicated that this process, when operated within appropriate ranges of these parameters, could potentially maximize nitrogen removal.

- The range of CT for optimum level of nitrogen reduction was found to be 2-hr to 3-hr and either increase in CT beyond 3-hr or decrease below 2-hr affected the removal efficiency.

- The most effective range of anoxic time fraction was predicted as 0.4 to 0.5 (i.e. 40 to 50%) of total cycle time. It was indicated that anoxic time fraction should be long enough so that all available NO_3^--N from aerobic phase was utilized prior to the completion of anoxic phase for effective denitrification. Also, on the other hand, the aerobic time fraction needed to be short so that exhaustion of NH_3-N concentration did not approach or fall below the ammonia half reaction constant for nitrification (K_{NH}) suppressing nitrification.

- Total nitrogen removal was in the order of 80 – 90% over 12 to 16-hr HRT for the wastewater composition selected.

The process performance described above was observed to correspond to a specific power input of 24.71 kW (50 kW/10^3 m^3) to the reactor and 15-day SRT with marginal variation for SRT values of either 20 or 25-day.

Similar results were found for running the same model with diffused aeration system with an air flow of 16.48 m^3/min (app. 33 m^3/min/10^3 m^3) into the tank. Predictions by the model had been found to agree with other study findings to a limited extent. Type and behavior of microbial population prevailing in the reactor and other external factors might affect process performance and oxygen diffusivity in either surface or diffused aeration systems. It was perceived that it might be required to suitably calibrate and optimize such intermittent aeration type activated sludge process depending on these factors on a case by case basis.

Critical Parameters for Nitrogen Removal and Suggested System Modifications

A number of important parameters for effective biological nitrogen removal were identified in the above process simulations. The most critical operating parameters recognized are listed in the following. Necessary modifications proposed for scaling these systems down to make them applicable for on-site wastewater treatment serving a single home has been discussed thereafter.

Influent BCOD:TKN Ratio

Influent ratio of biodegradable COD and total kjeldahl nitrogen (TKN) was noted to be an important determinant of biological nitrogen reduction in both the above techniques. It was predicted that COD-reducing heterotrophs being quick growers, sufficient biologically degradable organics should be present in the incoming wastewater to drive heterotrophic denitrifiers competing in the system for the same substrate. Such competition was more prevalent in SND process, where COD reduction and denitrification are predicted to occur concurrently, in comparison to intermittent aeration system where these two processes were separated in time. In these simulations, such a ratio was adopted as 10 that proved to be sufficient for significant nitrogen removal. Non-availability of organics is found to adversely affect extent of nitrogen removal.

It is assumed that, in possible application of these techniques to on-site wastewater treatment for nitrogen removal, sufficient organics (electron donor) will be available in the influent wastewater contained in the aeration tank over a reasonable period of time and it would not be necessary to supply any external carbon source. Endogenous decay of biomass within the system will also serve as a potential source of carbon for the active biomass.

Combination of SRT and Operating DO

Finding an effective combination of operating DO level and SRT was required for occurrence of SND process. The level of operating DO should not be too high and too low for adversely affecting anoxic denitrification and aerobic nitrification processes, respectively. Rather, it had to be selected to support and maintain these two processes simultaneously inside the same reactor. Based on the simulations performed, it was found that combination of 0.4 mg/L DO and 15-day SRT resulted in effective nitrogen removal.

To apply such technique in on-site wastewater treatment, the aerator of ATU has to be fitted with a PLC regulator that will have a DO measuring probe or device. This device would measure the operating DO level inside the reactor; send this information to the PLC, which in turn would adjust the aerator to operate at the required DO level.

Anoxic-Aerobic Cycle Time and Anoxic Time Fraction

In achieving nitrogen removal by intermittently aerated (IA) process, it was necessary to select an appropriate cycle time and strike the balance between the anoxic and aerobic fractions in order to effectively utilize the alternative cycle times. The optimum ranges of cycle times were 2 to 3-hr and the desirable range of anoxic time fraction was found as 40 to 50% (with other factors as; 15-day SRT, 12-hr HRT, and 0.5 sludge recycle ratio).

In applying this technique, a simple timer-based aeration system for the ATU and a separate submerged mixing device will be required. The mixing device will be to keep the biomass in suspension over the duration of anoxic cycle when the aerator is turned off. The timer can be pre-programmed to intermittently turn on and turn off the aeration unit to alternatively create anoxic and aerobic conditions inside the reactor.

Hydraulic Retention Time (HRT) and Sludge Recycle Ratio (R)

As per the model predictions, with other operating conditions being conducive for nitrogen removal, increases in either HRT or sludge recirculation ratio did not have a significant effect on process performance after adopting certain minimum values (12-hr HRT and 0.5 R, respectively) for these parameters.

As per the current regulations, the ATU tank is sized taking more than 24-hr detention time (MDOH, 2004) for the incoming flow which adequately satisfies this requirement. As the sludge is contained and retained inside the ATU reactor, the necessary recycle is also maintained. Hence, no modification to the way the ATU unit currently performs is perceived to be necessary in adjustment of these two parameters.

Solids Retention Time (SRT)

Adopting an adequate amount of SRT was found critical in terms of rendering enough opportunity for growth of the nitrifiers which are characteristically slow growers when compared to their heterotrophic counterparts. Nitrification being a precondition for denitrification, it became essential in biological nitrogen removal to promote nitrification for conversion of available NH_3-N to NO_3^--N. Providing an appropriate SRT in such a process was thus important to ensure growth of sufficient active nitrifiers inside the system. The two systems studied here were predicted to reduce appreciable amount of nitrogen when operated at 15-day SRT with other operating conditions as mentioned.

As no sludge wasting is currently being done from the ATUs, these units have a very large SRT value providing ample time for growth for the slow growing biomass. A large SRT is conceived to have little effect on the proposed operation of the system. As the sludge is not wasted form the tank, the endogenous decay of the active biomass will also take place inside the tank. The oxygen requirement for the unit will have two separate fractions, one for substrate removal or associated biomass growth and the other for decay of biomass. Initially, percentage of active biomass will be a significant portion of the total biomass in comparison to the decaying biomass. As the system will continue to operate, SRT will increase and the decaying biomass contained inside the system will start to increase as percentage of solids. Yet, with the fresh substrate coming in continuously, the portion of active biomass will be able to support its growth, though its proportion will decrease over time. Anoxic growth of nitrifiers is also maintained by suggested modification in operation of aeration system. Over time, the active and decaying biomass fractions will stabilize as will the oxygen requirement with this (Grady, 1999).

With no sludge wasting proposed, there will be two possible sinks for the nitrogen contained in the system, either conversion to nitrogen gas (N_2) by denitrification by virtue of the modified aeration system operation or with the effluent. First one is the desirable pathway of nitrogen removal being attempted here by proposing certain modifications, while the latter is left for the disposal system. A reasonable amount of detention time provided to the influent in the aeration tank will ensure enough time for heterotrophic removal of organics and negate the chance of washing out of active biomass with the effluent. Eventually, it is expected that the system will reach a steady state condition for combined removal of COD and a significant portion of nitrogen by promoting denitrification.

CHAPTER VII

CONCLUSIONS AND RECOMMENDATIONS

Performance evaluation of four types of on-site wastewater disposal techniques and possible system modification for enhanced nitrogen removal were studied in this research initiative. Of these four systems, the rock plant filter technique was eliminated during site selection because every system available to the study was found to be improperly performing. The remaining three types of on-site disposal techniques, once installed in appropriate subsurface soil conditions and having properly operating treatment system, were observed to achieve acceptable performances and eliminated as possible sources of non-point pollution. Since effluent standards were not available for these disposal techniques, this evaluation was based upon assumed discharged limits (e.g. 30 mg/L BOD, 5 mg/L TKN, and 200 cfu/100 ml). The level of treatment achieved in these disposal fields was observed to remain relatively unaffected by change in weather conditions. Performance did not increase significantly with depth of sampling certifying no change in existing regulations of compliance depth is needed. Therefore, it is concluded that these systems do not pose a potential threat to groundwater quality as the disposal fields were not seen to be contributing any appreciable amount of pollution load to groundwater.

Potential nitrogen removal by suggesting modifications to the biological treatment system in on-site wastewater management was investigated to restrict transport of nitrogen to the environment. This phase of the study was undertaken in anticipation of increasingly stricter regulations regarding the amount of nitrogen that could be discharged. Such an analysis was performed by altering the operation of aeration system of a conventional activated sludge system in a model of a suspended growth process to support effective nitrogen reduction by biological denitrification.

A summary of inferences drawn from the field data collected and modifications suggested as per predictions of the biological simulation model are given in the following sections.

On-site Disposal Conclusions

Rock Plant Filters

Rock plant filters were excluded from this performance evaluation study and it has been concluded that this type of disposal technique is not a feasible alternative for on-site wastewater management in the Mississippi coastal areas. Despite several visits made to the project area, not a single unit was observed to be working properly for its inclusion in this study. Most of the sites visited had a similar problem; wastewater surcharge to the ground surface that was due to plugging of pores by plant roots, wrong selection of plants, extended rooting system of the plants clogging the inlet and outlets, and improper plant maintenance.

Compliance to Discharge Limits

The C_{95} concentration values calculated for the two drip irrigation type system (site 1 and 2) conformed to the set discharge limit, except that for BOD level in one site (site 1). A closer review of the data set revealed that there had been only one instance where the BOD concentration had been high which was critical in working out the C_{95} concentration and threw the value out of the compliance criteria. Analysis of this second system (site 2) was based on a more exhaustive data points as compared to that of the first system to validate the findings. In view of this it was concluded that drip irrigation systems were operating acceptably. One of two sprinkler irrigation systems (site 4) was found to be working in conformity to the discharge limits. Excess concentrations found in case of one such system (site 3) in terms of higher BOD and NH_3–N concentrations could be attributed to non-functioning of aerator unit in aeration tank. The two mound systems adequately met the performance evaluation criteria at compliance depth over the entire set of weather conditions studied.

Subsurface Soil Characteristics

The soil sample analyses revealed that drip irrigation sites studied had subsurface soil condition varying from medium-fine to loamy sand; for sprinkler irrigation sites it was sandy loam, loamy sand, and silt loam; for mound site it ranged from high to low plasticity clay, loamy sand, and sandy loam. Installation of drip and sprinkler irrigation systems over disposal field having predominantly sandy soil and mound system over field underlain by mostly clayey soil were complying with the existing MDH regulations.

Effect of Weather Conditions

Level of treatment achieved by various disposal techniques was generally found to be unaltered by weather conditions, e.g. cold, dry, wet, warm, and also their interactions. However, in a few cases, significant difference was actually observed. It was attributed to non-functioning of treatment unit of the disposal system. Otherwise, though variation was found, the parameter value was observed to be within discharge limits and as such did not contribute any potential pollutant load to the environment.

Modification to Existing Regulations

On analyzing the results, if attention was drawn specifically to those cases where the discharge parameters were not meeting the limits, the reason for significant variation in treatment level could be correlated either to improper functioning of the treatment system (site 3) or insufficient data to adequately justify and conclude the findings by statistical analysis (site 1). From such findings the reason for non-compliance to specified limits could not be traced back to inadequate functioning of the disposal field. A few parameters, despite showing variation in concentrations over depth, were observed to be satisfying the discharge limits at compliance level. Apart from these isolated instances, all types of disposal techniques evaluated showed that the systems were working properly meeting the set limits at compliance depth. This negated any modifications to the existing regulations of adding an extra foot to the compliance depth.

Groundwater Quality

The study also investigated the possibility of groundwater contamination from these disposal fields due to their inadequate functioning. In most of the cases, no statistically significant difference could be found between pollutant concentrations in groundwater samples across the disposal fields. Even when such differences were detected for a few of the parameters, the field sampling results confirmed that the disposal system was meeting the stipulated discharge limits for those parameters. This eliminated the possibility of relatively increased levels of pollutants being contributed by insufficient treatment to percolating wastewater by the disposal fields. Higher concentrations of COD in a couple of the sites were observed to be due to the pre-existing conditions of soil matrix and could not be categorically related to improper functioning of disposal systems.

Treatment Improvement Conclusions

As per the model forecasts, two separate modifications in efficient management of aeration system of the ATU were proposed to enhance biological denitrification. The first one was operation of the aeration system at low DO concentration by controlling the aerator with the help of a sensor device that could measure DO. This arrangement was expected to promote a condition where heterotrophic COD removal, autotrophic nitrification, and heterotrophic denitrification could occur simultaneously.

The other option proposed was to use intermittent aeration by a timer based aerator control to alternatively create anoxic and aerobic conditions inside the aerobic reactor to separate anoxic denitrification from the other two processes requiring aerobic conditions.

Recommendations

Though qualified but conclusive remarks could be made about those specific systems and parameters for which this evaluation study was conducted, it was recommended that such studies should be continued for other available types of on-site disposal systems under different field settings. It was only by conducting a number of such studies with diverse systems under varying conditions over a reasonable period of time that a workable data base could be obtained to substantiate their working capability and adherence to regulations.

One particular aspect that was not taken under the purview of the present study was measurement of nitrate-nitrogen (NO_3^--N). It seemed that escape of nitrogen in this form might not have been a problem where anaerobic treatment was used prior to disposal fields. But, in case of systems employing aerobic treatment, measuring NO_3^--N could have given a better picture if there was any significant portion of total nitrogen being lost in this form and not captured by the disposal field. Hence, it was suggested that this particular nitrogen parameter should be included in future evaluation studies.

The model predictions for biological nitrogen removal showed encouraging results but these results and proposed operation of the systems needed to be verified experimentally to define the range over which optimum nitrogen reduction could be achieved under a practical scenario. The operating conditions that were indicated in the analysis depended on several external factors. Predictions put forward by the modeling and simulations done were required to be established and reaffirmed by experimental studies before these findings could be conclusively proved. Such a research initiative could be taken up first by conducting laboratory scale studies. On obtaining expected and promising results from such analysis, trials in the field would be taken up further to replicate actual scenario.

One important aspect of these predictions made in simulating these two potentially effective biological nitrogen removal processes was that these apparently efficient aeration management techniques could be utilized for possible use in centralized wastewater treatment systems also. Future research on this could be directed towards justifying these findings and predictions by conducting laboratory scale or pilot plant studies. Thus, if the extent of nitrogen removal as forecast by these simulations with simple modifications to aeration system could be established and validated experimentally, that would prove to be very important and effective from the standpoint of achieving nitrogen reduction in municipal wastewater treatment with activated sludge systems that might be suitably retrofitted to serve this purpose.

BIBLIOGRAPHY

American Society for Testing and Materials. (1998). "Standard Test Method for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass" Annual Book of ASTM Standards, ASTM International, West Conshohocken, PA., ASTM D 2216-98

American Society for Testing and Materials. (2005). "Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils" Annual Book of ASTM Standards, ASTM International, West Conshohocken, PA., ASTM D 4318 - 05

American Society for Testing and Materials. (2006). "Standard Test Methods for Amount of Material in Soils Finer than No. 200 (75-μm) Sieve" Annual Book of ASTM Standards, ASTM International, West Conshohocken, PA., ASTM D 1140 - 00

Applegate C. S., Wilder B., Deshaw J. R. (1980). "Total nitrogen removal in a multi-channel oxidation system". *Journal of Water Pollution Control Federation,* 52, 568-577

Beggs R. A., Tchobanoglous T., Hills D., Crites R. W. (2004). "Modeling subsurface drip application of onsite wastewater treatment system effluent", *Proceedings of the 10th National Symposium on Individual and Small Community Sewerage Systems*, American Society of Agricultural Engineers, Sacramento, CA, March 21-24, 92-103

Bertanza G. (1997). "Simultaneous nitrification-denitrification process in extended aeration plants: pilot and real scale experiences". *Water Science and Technology* 35(6), 53-61

Bilanovic D., Battistoni P., Cecchi F., Pavan P., Mata-Álvarez J. (1999). "Denitrification under high nitrate concentration and alternating anoxic conditions". *Water Research*, 33, 3311-3320

Bouma, J. (1975). "Unsaturated flow during soil treatment of septic tank effluent". *Journal of Environmental Engineering Division*, American Society of Civil Engineers, 101, 967-983

Bradley B. (2009). "California tackles nitrogen from onsite wastewater systems". *Southwest Hydrology,* July-August, 2009

Choubert J. M., Stricker A. E., Marquot A., Racault Y., Gillot S., Héduit A. (2009a). "Updated activated sludge model n^{o} 1 parameter values for improved prediction of nitrogen removal in activated sludge processes: validation at 13 full-scale plants". *Water Environment Research*, 81(9), 858-865

Choubert J. M. (2009b). Personal communications. November 2009

Copp J. B., Spanjers H., Vanrolleghem P. A. (2002). "Repository in control of the activated sludge process: Benchmarking control strategies". *IWA Scientific and Technical Report No. 11*. London, UK

Cox C. D. (2004). "Statistical distribution of uncertainty and variability in activated sludge model parameters". *Water Environment Research*, 76(7), 2672-2685

Daigger G. T., Littleton H. X. (2000). "Characterization of simultaneous nutrient removal in staged, closed-loop bioreactors". *Water Environment Research* 72(3), 330-339

Drews R. J. L. C., Greeff A. M. (1973). "Nitrogen elimination by rapid alteration of aerobic/ anaerobic conditions in Orbal activated sludge plants". *Water Research,* 7, 1183-1194

Drews R. J. L. C., Malan W. M., Meiring G. J., Moffatt B. (1972). "The Orbal extended aeration activated sludge plant". *Water Research,* 44(2), 221-231

Eastburn R. P., Ritter W. F. (1984). "Denitrification in on-site wastewater treatment systems – A review", *Proceedings of the Fourth National Symposium on Individual and Small Community Sewerage systems*, American Society of Agricultural Engineers, New Orleans, LA, December 10-11, 305-313

Firestone M. K. (1982). "Biological denitrification – Nitrogen in agricultural soil", *J Stevenson ed. Monograph No. 22*, American Society of Agronomy, Madison, WI, 289-326

Grady Jr. C. P. L., Daigger G. T., Lim H. C. (1999). "*Biological Wastewater Treatment*", 2nd ed. New York: Marcel Dekker, Inc.

GPS-X, Software for Modeling and Simulation of Municipal and Industrial Wastewater Treatment Processes, Hydromantis Inc., Ontario, Canada

Habermeyer P., Sánchez A. (2005). "Optimization of the intermittent aeration in a full-scale wastewater treatment plant biological reactor for nitrogen removal". *Water Environment Research*, 77(3), 229- 233

Hanhan O., Artan N., Orhon D. (2002). "Retrofitting activated sludge systems to intermittent aeration for nitrogen removal". *Water Science and Technology*, 46(8), 75-82

Héduit A., Duchene P. H., Sintes L. (1990). "Optimisation of nitrogen removal in small activated sludge plants". *Water Science and Technology*, 22(3-4), 123-130

Henze M., Grady C. P. L. Jr., Gujer W., Marais G. v. R., Matsuo T. (1987a). "*Activated sludge model no. 1*". IAWPRC Scientific and Technical Report, No. 1, International Association on Water Pollution Research and Control, London

Henze M., Grady Jr. C. P. L., Gujer W., Marais G. v. R., Matsuo T. (1987b). "A general model for single-sludge wastewater systems". *Water Research,* 21, 505-515

Hidaka T., Yamada H., Kawamura M., Tsuno H. (2002). "Effect of dissolved oxygen conditions on nitrogen removal in continuously fed intermittent-aeration process with two tanks". *Water Science and Technology,* 45(12), 181-188

Holman J. B., Wareham D. G. (2005). "COD, ammonia and dissolved oxygen time profiles in the simultaneous nitrification/ denitrification process". *Biochemical Engineering Journal,* 22, 125-133

Insel G., Artan N., Orhon D. (2005). "Effect of aeration on nutrient removal performance of oxidation ditch systems". *Environmental Engineering Science,* 22(6), 802-815

Insel G., Sin G., Lee D. S., Nopens I., Vanrolleghem P. A. (2006). "A calibration methodology and model-based system analysis for nutrient removing SRBs under reduced aeration". *Journal of Chemical Technology and Biotechnology,* 82(4), 679-687

Jobbágy A., Simon J., Plósz B. (2000). "The impact of oxygen penetration on the estimation of denitrification rates in anoxic processes". *Water Research*, 34, 2606-2609

Kansas State University Agricultural Experiment Station and Cooperative Extension Service. (1998). "Rock-Plant Filter Design and Construction for Home Wastewater Systems", Kansas State University

Lessard P., Tusseau-Vuillemin M-H., Héduit A., Lagarde F., (2007). "Assessing chemical oxygen demand and nitrogen conversions in a multi-stage activated sludge plant with alternating aeration". *Journal of Chemical Technology and Biotechnology*, 82, 367-375

Louzeiro N. R., Mavinic D. S., Oldham W. K., Meisen A., Gardner I. S. (2002). "Methanol-induced biological nitrogen removal kinetics in a full-scale sequencing batch reactor". *Water Research*, 36, 2721-2732

Lucasse L. J. S., Keesman K. J. (1999). "Optimised operation and design of alternating activated sludge process for N removal". *Water Science and Technology*, 33(11), 2651-2659

Lukasse L. J. S., Keesman K. J., Klapwijk A., Straten G. (1998). "Optimal control of N-removal in ASPs". *Water Science and Technology,* 38(3), 255-262

Münch E. V., Lant P., Keller J. (1996). "Simultaneous nitrification and denitrification in bench-scale sequencing batch reactors". *Water Research,* 30(2), 277-284

Middlebrooks E. J. (1995). "Upgrading pond effluents: an overview". *Water Science Technology*, 31, 353-368

Middlebrooks E. J. (1988). "Review of rock filters for the upgrade of lagoon effluents", Journal of Water Pollution Control Federation, 60, 1657-1662

Minitab, Statistical Analysis Software, Minitab Inc. Pennsylvania, USA

Mississippi Department of Health. (2006). "Regulations Governing On-site Wastewater Disposal: Design Standard VI – Spray Irrigation Disposal System", Mississippi Department of Health, Jackson, MS

Mississippi Department of Health. (2004). "Regulations Governing On-site Wastewater Disposal: Design Standard IA – Septic Tanks, IB- Aerobic Treatment Units", Mississippi Department of Health, Jackson, MS

Mississippi State Department of Health. (2002). "Repair of Failing On-site Wastewater Systems", *Bureau of General Environmental Services, Division of Onsite Wastewater,* Jackson, MS

Mississippi Department of Health. (1997a). "Regulations Governing On-site Wastewater Disposal: Design Standard V - Elevated Sand Mound Disposal System", Jackson, MS

Mississippi Department of Health. (1997b). "Regulations Governing On-site Wastewater Disposal: Design Standard IV – Subsurface Drip Disposal System", Mississippi Department of Health, Jackson, MS

Mississippi Department of Health. (1997c). "*Regulations Governing On-site Wastewater Disposal: Design Standard VII – Plant Rock Filter*", Mississippi Department of Health, Jackson, MS

O'Brien W. J., McKinney R. E., (1979). "Removal of lagoon effluent suspended solids by a slow rack filter". U.S. Environmental Protection Agency. Municipal Environmental Research Laboratory, Cincinati, Ohio [EPA Document No. EPA-600-2-79-011, NTIS No. PB 27454]

Oh J., Silverstein J. (1999). "Oxygen inhibition of activated sludge denitrification". *Water Research*, 33, 1925-1937

Otis R. J., Boyle W. C., Sauer D. R. (1974). “The performance of household wastewater treatment units under field conditions”, *Proceedings of the national home sewage disposal symposium*, American Society of Agricultural Engineers, St. Joseph, MI, 191-201

Perkins R. J. (1989). “*On-site Wastewater Disposal*”, Lewis Publishers: Chelsea, Michigan

Randall C. W., Barnard J. L., Stensel H. D. (1992). “*Design and retrofit of wastewater treatment plants for biological nutrient removal*”. Water Quality Management Library, Vol. 5. Lancaster PA: Technomic Publishing Co.

Rich L. G. (1988). “A critical look at rock filters”, *Journal of Environmental Engineering*, American Society of Civil Engineers, 114, 219-223

Rittmann B. E., Langeland W. E. (1985). “Simultaneous denitrification with nitrification in single-channel oxidation ditches”. *Journal of Water Pollution Control Federation,* 57, 300-308

Rittman B. E., McCarty P. L. (2001). “*Environmental Biotechnology: Principles and Applications”*. Mc-Graw Hill, New York

Sasaki K., Yamamoto Y., Tsumura K., Ouchi S., Mri Y. (1996). “Development of 2-reactor intermittent-aeration activated sludge process for simultaneous removal of nitrogen and phosphorus”. *Water Science and Technology*, 34(1-2), 111-118

Sliekers A. O., Derwort N., Gomez J. L. C., Strous M., Kuenen J. G., Jetten M. S. M., (2002). “Completely autotrophic nitrogen removal over nitrite in one single reactor”, *Water Research*, 36, 2475-2482

Sikora L. J., Corey D. R. (1976). “Fate of nitrogen and phosphorus in soils under septic tank disposal fields”. *Trans. of the American Society of Agricultural Engineers*, 19, 866-870

Sikora L. J., Keeney D. R. (1974). “Laboratory studies on simulation of biological denitrification”, *Proceedings of the National Home Sewage Treatment Symposium*, Dec. 9-10, Chicago, IL, American Society of Agricultural Engineers Pub. PROC-175, 64-73

Sin G., Insel G., Lee D. S., Vanrolleghem P. A. (2004). “Optimal but robust N and P removal in SRBs: A systematic study of operating scenarios”. *Water Science Technology,* 50(10), 97-105

Soil Texture Triangle, defined by the USDA, <http://en.wikipedia.org/wiki/Soil_texture> October 1, 2009

Sowers G. B., Sowers G. F. (1970). "Introductory Soil Mechanics and Foundation", 3rd Ed., The Macmillan Company, London, 71-73

Standard Methods (1998). "Standard Methods for the Examination of Water and Wastewater". 20th Ed. American Public Health association, Washington D.C.

Statistical Analysis Software, SAS Institute, Inc. North Carolina, USA

Stewart L. W., Carlile B. L., Cassell D. K. (1979). "An evaluation of alternative simulated treatments of septic tank effluent", *Journal of Environmental Quality*, 10, 528-531

Stewart L. W., Reneau Jr. R. B. (1984). "Septic tank effluent disposal experiments using nonconventional systems in selected coastal plain soils", *Final report to Virginia Dept. of Health*, Virginia Polytechnique Institute and State University, Blacksburg, VA

Surmacz-Górska J., Cichon A., Miksch K. (1997). "Nitrogen removal from wastewater with high ammonia nitrogen concentration via shorter nitrification and denitrification". *Water Science and Technology*, 36(10), 73-78

Tchobanoglous G., Ruppe L., Leverenz H., Darby J. (2004). "Decentralized wastewater management: challenges and opportunities for the twenty-first century", Water Science and Technology: Water Supply, 4(1), 95-102

Tchobanoglous G., Crites R. W., (1998). "Small & Decentralized Wastewater Management Systems", McGraw-Hill; 1st Edition, New York

Tchobanoglous G, Stensel H D. (2002). "*Wastewater Engineering – Treatment & Reuse*". Metcalf & Eddy, Inc. 4th ed. Burton F L, Editor. New York: McGraw-Hill.

Tiedje J. M. (1988). "*Biology of Anaerobic Microorganisms*". In: A J B Zehnder, Editor. Ecology of denitrification and dissimilarity nitrate reduction to ammonium. New York: John Wiley. 179-244

U.S. Environmental Protection Agency. (2005). "Decentralized Wastewater Treatment Systems: A Program Strategy", US EPA Publications Clearinghouse, Cincinnati, Ohio [EPA Document No. 832-R-05-002]

U.S. Environmental Protection Agency. (2002). "Wastewater Technology Fact Sheet – Rock media Polishing Filter for Lagoons", [EPA Document No. 832-F-02-023], Office of Water Washington, D.C.

U.S. Environmental Protection Agency. (2000). "Decentralized Systems Technology fact Sheet: Aerobic Treatment", [EPA Document No. 832-F-00-031], Office of Water Washington, D.C.

U.S. Environmental Protection Agency. (1999). "Decentralized Systems Technology fact Sheet: Mound Systems", [EPA Document No. 832-F 99-074], Office of Water Washington, D.C.

U.S. Environmental Protection Agency. (1998). "National Water Quality Inventory: 1998 Report to Congress", [EPA Document No. 841-R-00-001], Office of Water Washington, D.C.

U.S. Environmental Protection Agency. (1997). "Response to Congress on Use of Decentralized Wastewater Treatment Systems", US EPA Publications Clearinghouse, Cincinnati, Ohio [EPA Document No. 832R97001b]

U.S. Environmental Protection Agency. (1979). "Removal of lagoon effluent suspended solids by a slow-rock filter", O'Brian W J, McKinney K E, [EPA Document No. 600/2-79-011]

Vanhooren H., Meirlean J., Amerlinck Y., Claeys F., Vangheluwe H., Vanrolleghem P. A. (2003). "WEST: Modeling biological wastewater treatment". *Journal of Hydroinformatics,* 5, 27-50

Walker W. G., Bouma J., Keeney. D. R., Olcott P. G. (1973). "Nitrogen transformations during subsurface disposal of septic tank effluent in sands: I. soil transformations", *Journal of Environmental Quality*, 2, 475-479

Whitmyer R. W., Apfel R. A., Otis R. J., Meyer R. L., (1991). "Overview of individual onsite nitrogen removal systems", *Proceedings of the Sixth On-Site Wastewater Treatment Conference.* American Society of Agricultural Engineering, St. Joseph, MI, 143-154

www.Groundwatersoftware.com website (2009). <http://www.ngwa.com/~gwater/Calculators/ calculator6.htm> (June13, 2009)

Zhao H. W., Mavinic D. S., Oldham W. K., Koch F. A. (1999). "Controlling factors for simultaneous nitrification and denitrification in a two-stage intermittent aeration process treating domestic sewage". *Water Research*, 33, 961-970

Zhao H. W., Mavinic D. S., Oldham W. K., Koch F. A. (1998). "Factors affecting phosphorus removal in a two-stage intermittent aeration process treating domestic sewage". *Water Science and Technology*, 38, 115-122

Zeng R. J., Lemaire R., Yuan Z., Keller J. (2003). "Simultaneous nitrification, denitrification, and phosphorus removal in a lab-scale sequencing batch reactor". *Biotechnology and Bioengineering,* 84(2), 170-178

APPENDIX A

ADDRESSES OF STUDY LOCATIONS

ADDRESSES OF STUDY LOCATIONS

Type of OSWWT in place	Site	Name and Address
Septic tank with Subsurface Drip Disposal Systems (Leaching fields)	Site 1	Ms. Mill Mott 808 Orange Street Ocean Springs, MS 39564 228-872-6218
	Site 2	Mr. Jeff Medlen 5704 South Street Ocean Springs, MS 39564 228-875-1757
Aerobic Treatment Unit (ATU) with Spray Irrigation Disposal Systems	Site 3	Mr. Dennis Griffin 2901 Hamill Farm Road Ocean Springs, MS 39564 Ph. 228-217-3009
	Site 4	Mr. Percy Guster 3213 Westlane Gautier, MS 39553-5845 Ph. (601) 618-5542
Septic Tank with Elevated Sand Mound Disposal Systems	Site 5	Mr. Hershel Richards 15486 Mark West Road Gulfport, MS 395 Ph. 228-539-1945
	Site 6	Cheryl and Gary Gage 7096 Woolmarket Biloxi, MS 39532 228.392.3151

APPENDIX B

SAMPLE SITE SURVEY QUESTIONNAIRE

MS Department of Health, MS Department of Environmental Quality, and MS State University
Evaluation of Coastal Subsurface Wastewater Disposal Systems
Site Survey Questionnaire

We appreciate your assistance in this project. The following questions will aid in characterizing both flow and composition of the wastewater being treated at this site.

Note: We plan to notify you at least you at least at least 48 hours before arriving your premises for sample collection by our research team.

It would be convenient for us if we could use one of your available outside power and water outlets during sampling. We will not be using absolutely anything more than 50¢ worth of electricity and a few gallons of water for cleaning purposes per sampling and probably much less than that actually.

Please let us know if you are willing to let us do that? Yes No

I. House

a. Contact Information

Site Address____ Zip Code ___________

GPS Coordinates: Latitude________ Longitude _________ Tel/Email ________________________

b. Do you own or rent your residence? Own Rent

c. Type of structure

Mobile home Pre-manufact. modular house Single-story frame Multi-story frame

d. Approximate age of the residence (in years):

< 5 years 5 to 9 years 10 to 14 years 15 to 19 years 20 + years

e. Approximate size of the residence (in square feet):

< 1,000 1,000 to 1,999 2,000 to 2,999 3,000 or larger

f. Number of Bedrooms within the residence:

One Two Three Four More than Four

g. Number of Bathrooms within the residence:

One Three More than Four

Two Four Residence also has half-bath

II. Occupancy

a. Total number of persons residing at this location:______________

b. Approximate ages of persons residing at this location:

	Age, years				
	< 17	18 to 35	36 to 53	54 to 71	72 +
Number residing at this location					

c. How many residents spend more than 16 hours per day at home:____________

III. Special Conditions

a. Do you have a dishwasher installed? Yes No

If yes, how often is it used?

Less than once per week	3 – 4 times per week	Once per day
1 – 2 times per week	5 – 6 times per week	More than once per day

b. Do you have a garbage disposal system installed? Yes No

c. Do you have a washer and dryer unit installed? Yes No

If yes, how often is it used?

1 day per week	3 days per week	5 days per week
2 days per week	4 days per week	Daily

Also, how often full loads are processed on a weekly basis (small loads are about one-half of the size of the full load)?

< 1 load 1 load 2 loads 3 loads More than 3 loads

IV. Wastewater Treatment and Disposal

a. Type of wastewater treatment system installed at this location:

Treatment	Disposal
Septic Tank	Sub-surface drainfield (leachfield)
Aerobic Treatment Unit (ATU)	Above-ground drainfield (Mound System)
	Spray field

b. Approximate age of installed treatment system:

< 5 years 5 to 9 years 10 to 14 years 15 to 19 years 20 + years

c. Approximate time since last maintenance on the installed treatment system:

No maintenance has been performed	1 to 3 years	5 to 10 years
< 1 year	3 to 5 years	> 10 years

What was done (briefly)? ____

Signature:__________________________ Date:_____________

APPENDIX C

SUMMARY OF SITE SURVEY INFORMATION

Appendix C

Summary of Site Survey Information

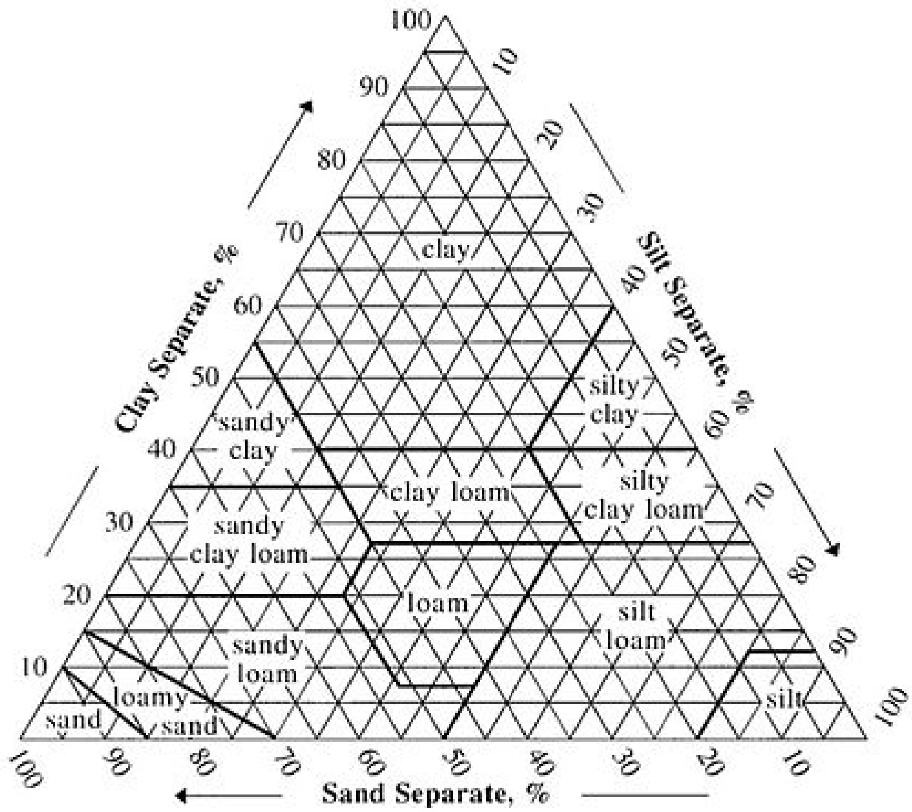
Item	Site 1	Site 2	Site 3	Site 4	Site 5	Site 6
Disposal System						
Type of Disposal System	Drip irrigation		Sprinkler irrigtn.		Sand mound	
Approximate age, range yr	10-14	> 20	5-9	5-9	< 5	10-14
App. time since last maintenance, range yr	1-3	5-10	1-3	1-3	None	3-5
Property						
Holding	owned	owned	owned	owned	owned	owned
Age of residence, range yr	10-14	> 20	5-9	5-9	< 5	< 5
App. Size, range sq ft	1,000-1,999	1,000-1,999	> 3,000	1,000-1,999	2,000-2,999	1,000-1,999
No of bedrooms	3	3	> 4	3	3	3
No of bath rooms	2	2	> 4	2	2	2
Occupancy						
Total number of persons	1	3	5	3	2	3
Residents spending > 16 hr per day at home	0	0	0	0	2	0
Installed Facilities						
Dish washer	√	×	√	√	√	×
Weekly use	1-2	×	1-2	1-2	3-4	×
Garbage disposal system	×	×	×	√	√	×
Washer and dryer unit	√	√	√	√	√	√
Weekly use	2	2	4	2	2	3
Full loads per week	3	3	> 3	1	2	2

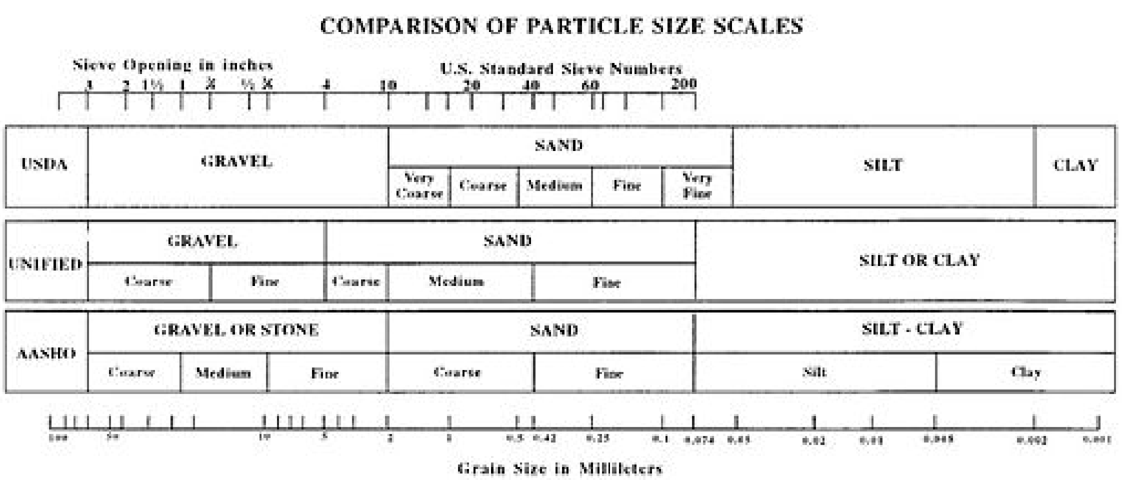
APPENDIX D

SOIL TEXTURE TRIANGLE

Appendix D

Soil Texture Triangle Showing the USDA Classification System based on Grain Size





Source: <http://en.wikipedia.org/wiki/Soil_texture>

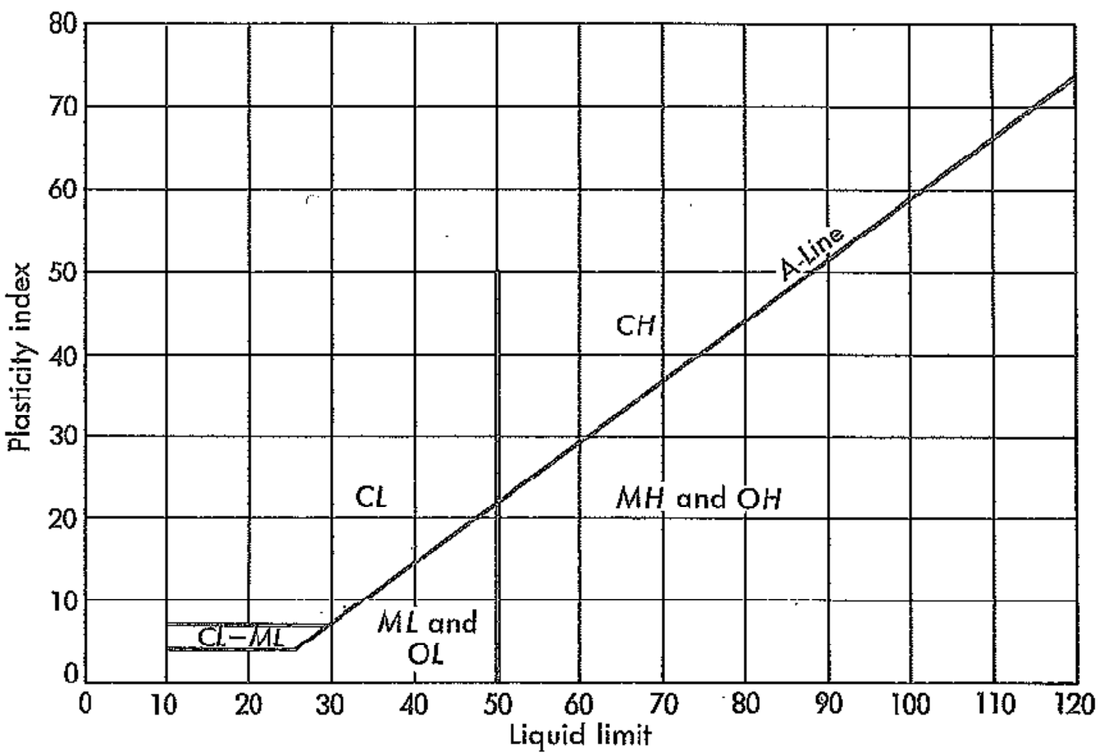
APPENDIX E

PLASTICITY CHART FOR THE CLASSIFICATION OF FINE-GRAINED SOIL

Appendix D.1

Plasticity Chart for the Classification of Fine-grained Soil

(Tests made on fraction finer than No. 40 sieve)

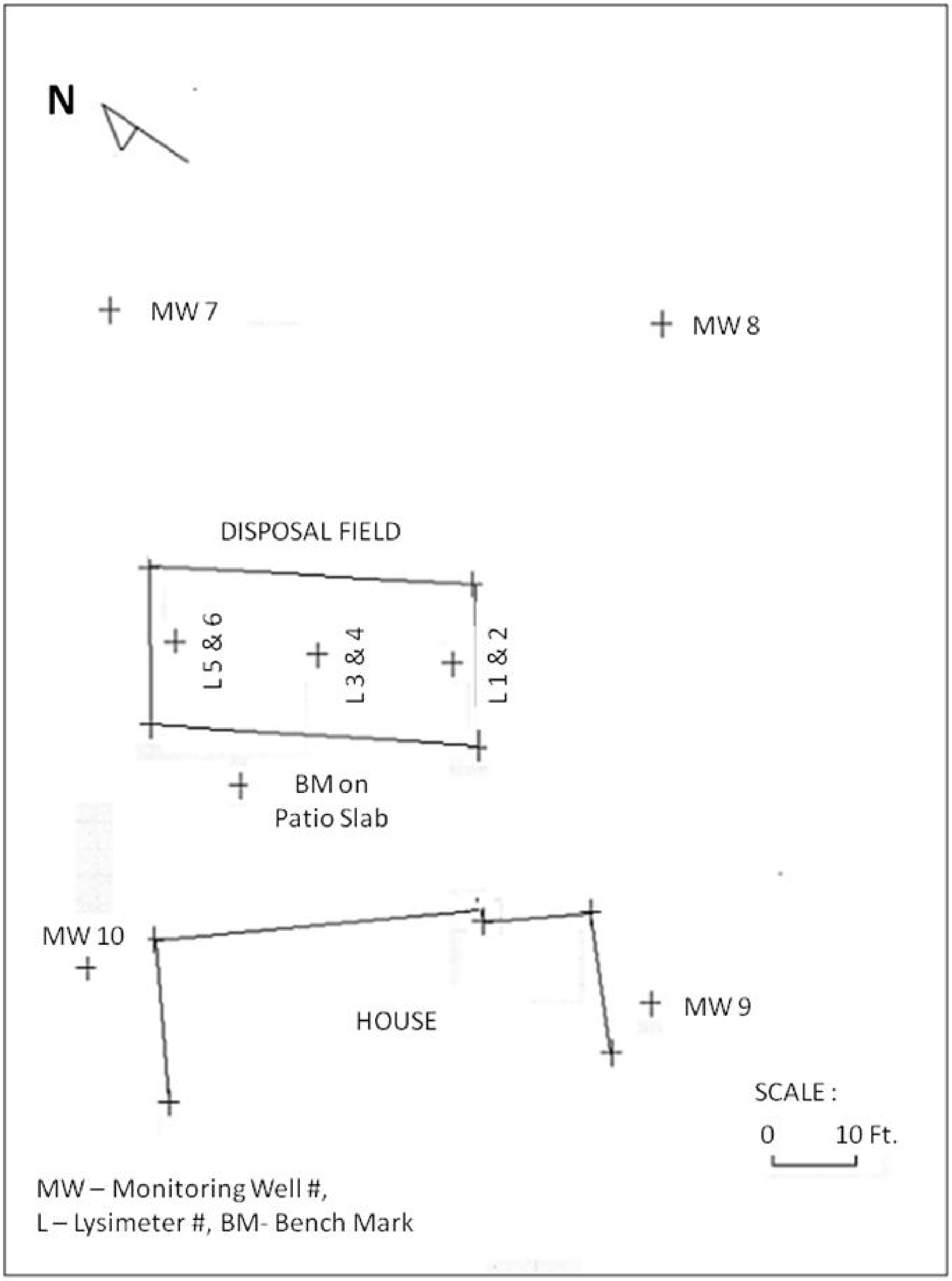


Source: Sowers, 1970

APPENDIX F

SITE LAYOUTS

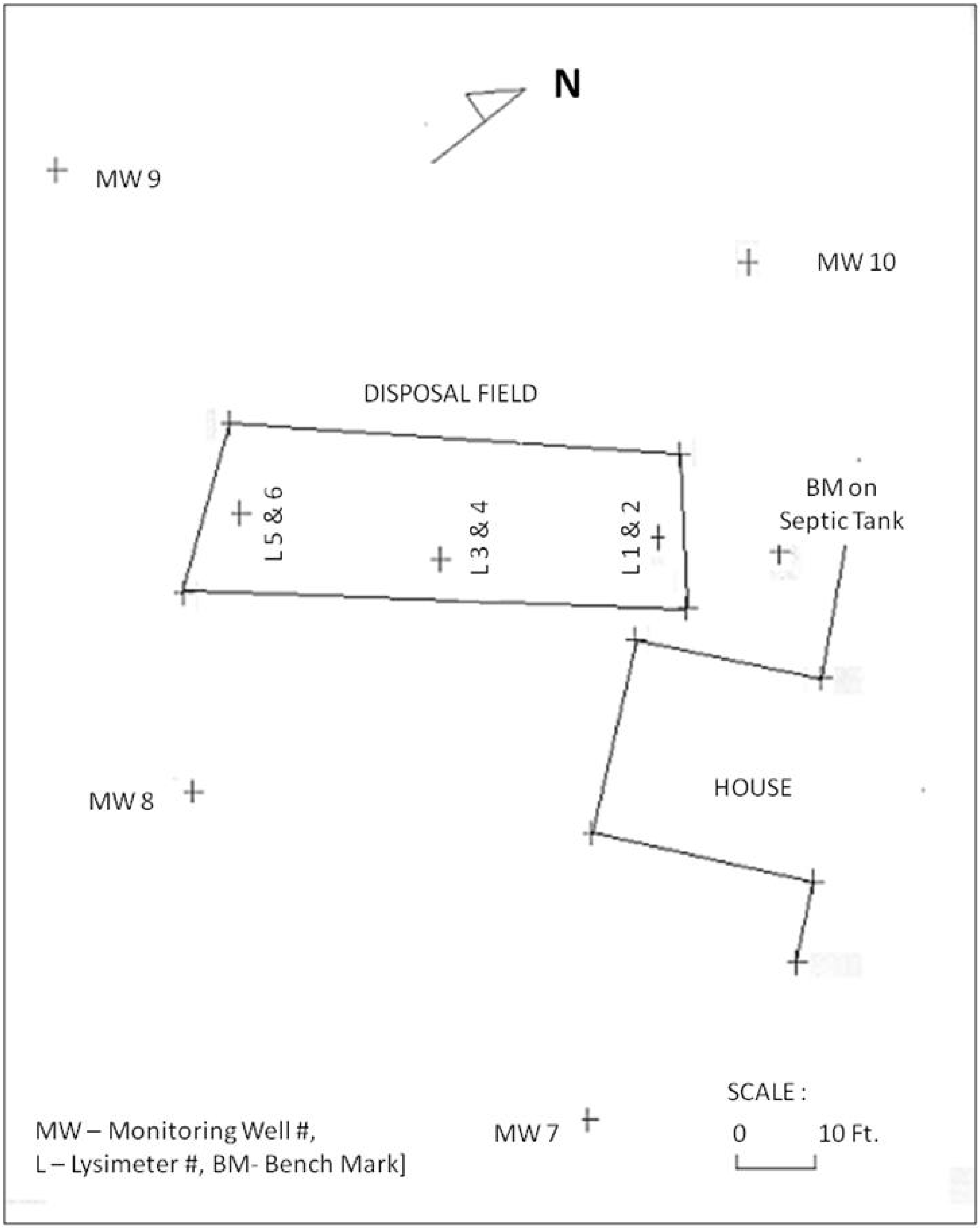
Appendix F.1 – Layout of Site 1 (Subsurface Drip Irrigation)



Site 1 : Drip Irrigation Site

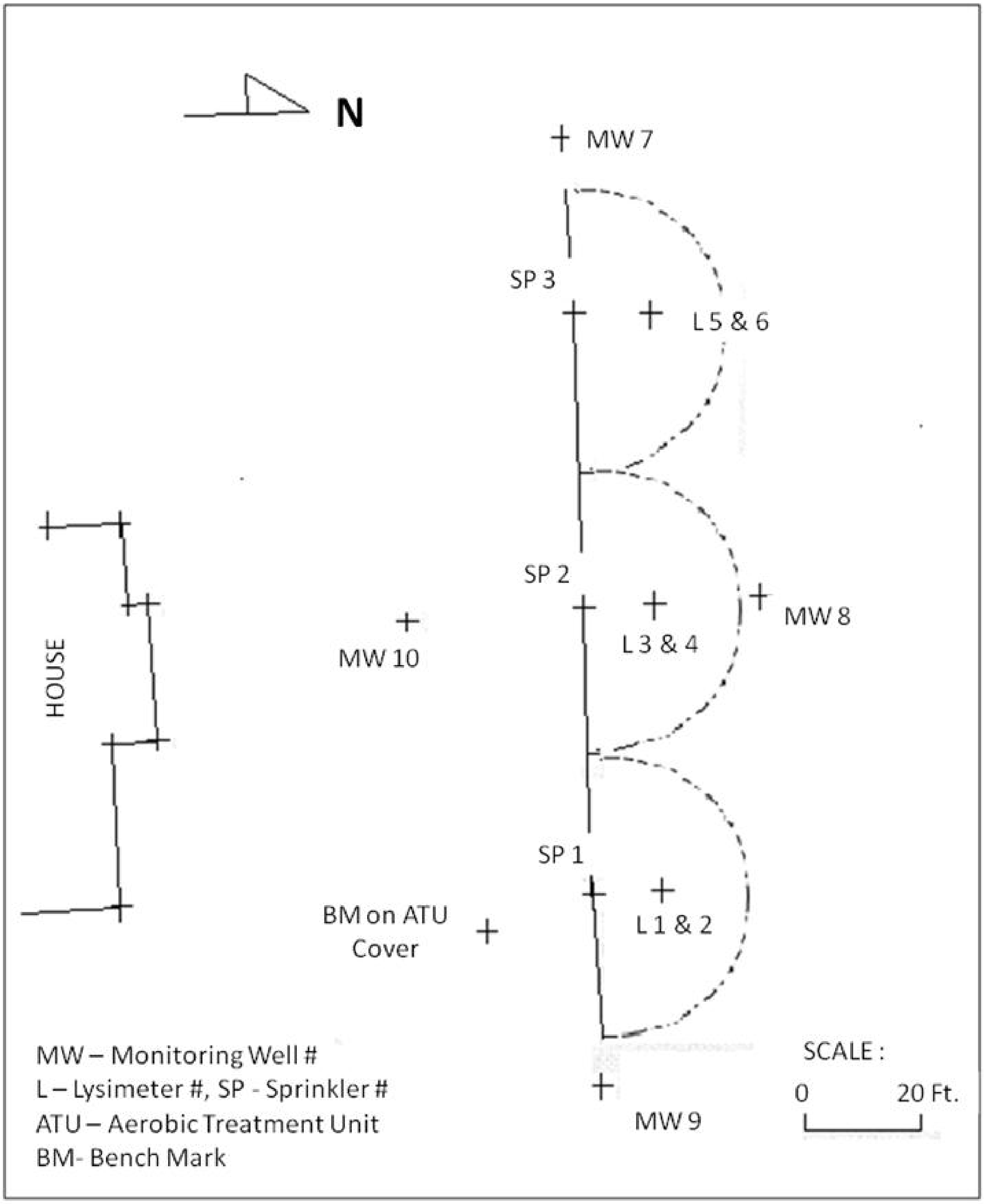
Address : 808 Orange Street, Ocean Springs, MS 39564

Appendix F.2 – Layout of Site 2 (Subsurface Drip Irrigation)



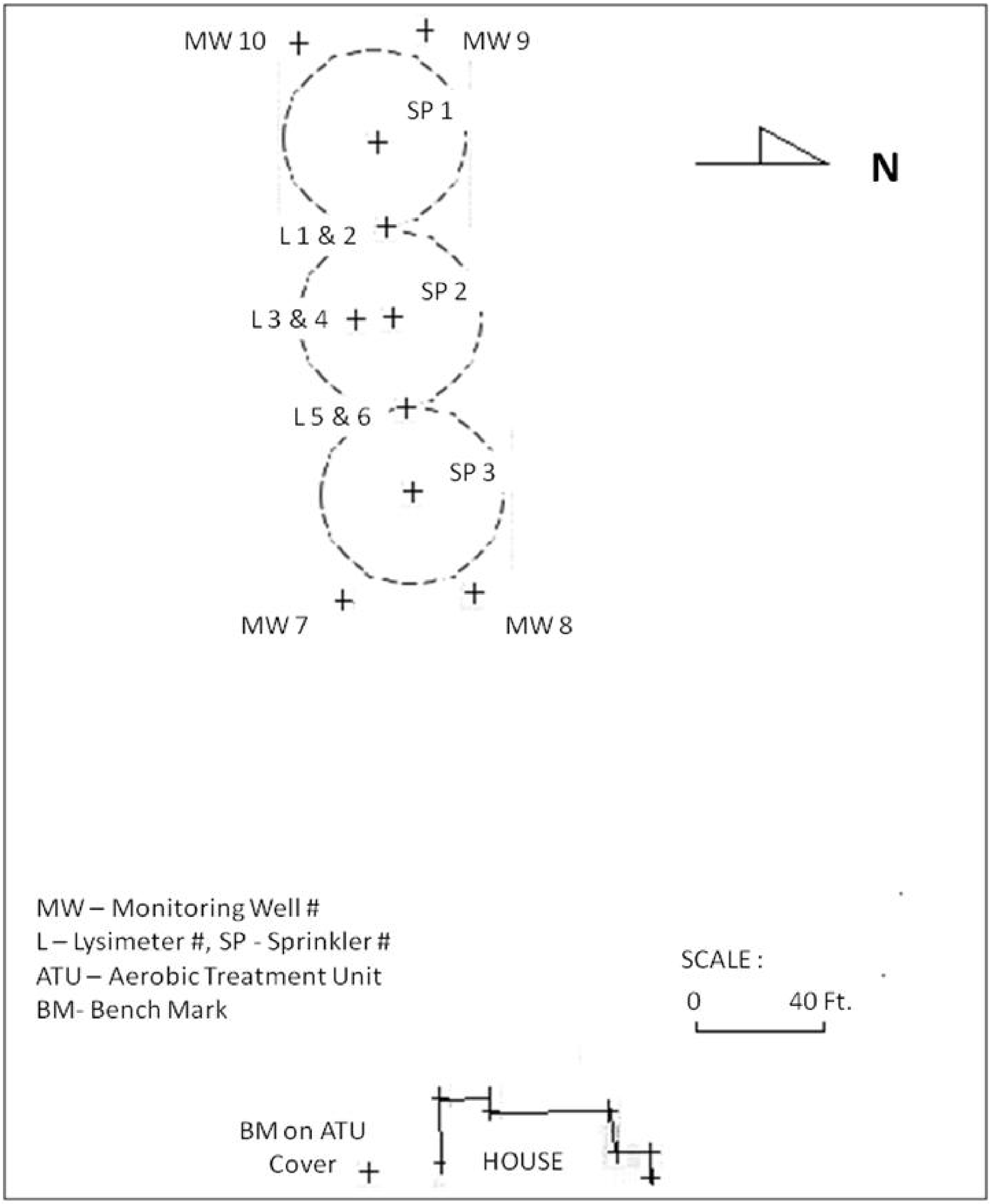
Site 2 : Drip Irrigation Site
Address : 5704 South Street, Ocean Springs, MS 39564

Appendix F.3 – Layout of Site 3 (Sprinkler Irrigation)



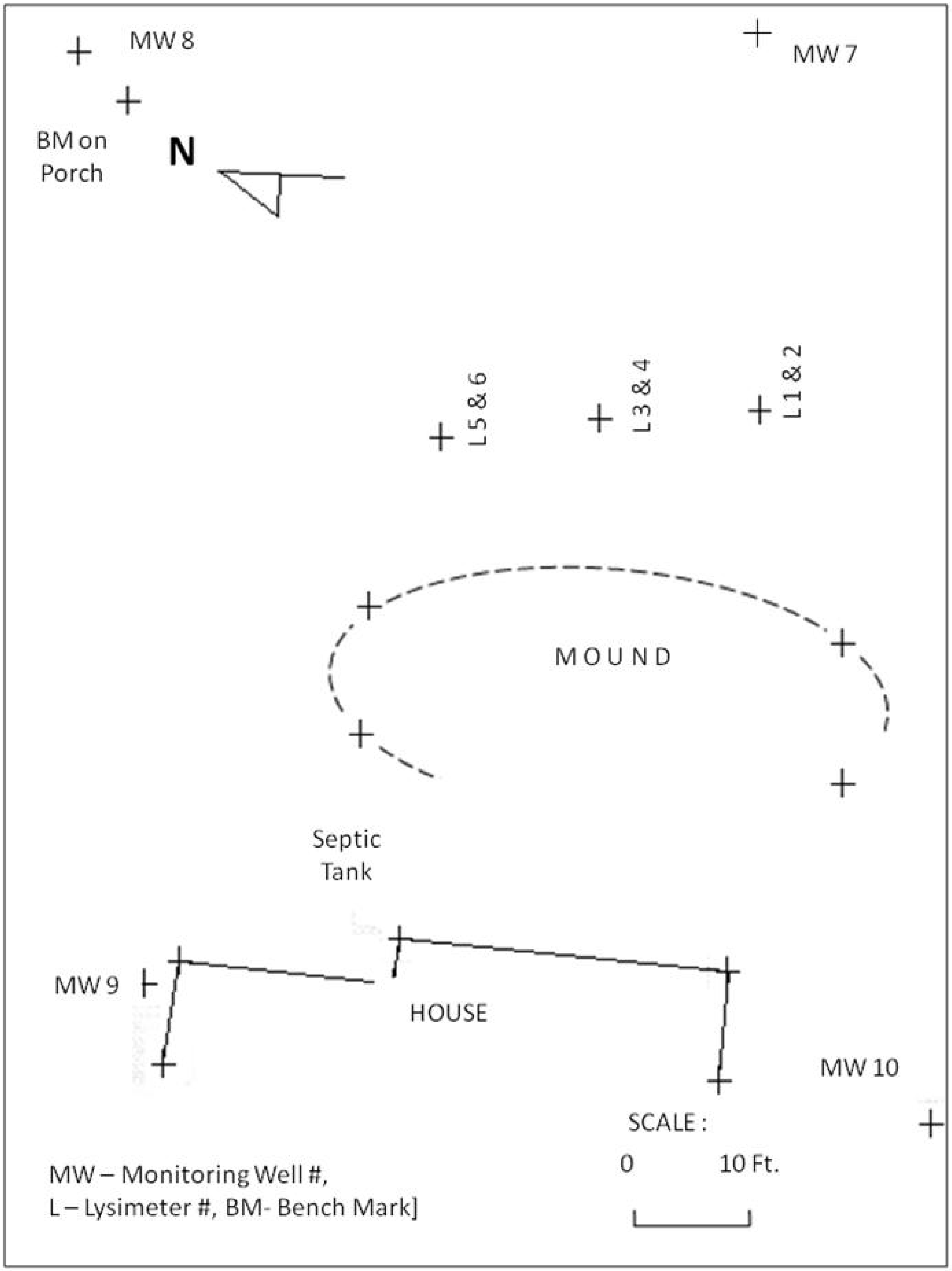
Site 3 : Sprinkler Irrigation Site
Address : 2901 Hamill Farm Road, Ocean Springs, MS 39564

Appendix F.4 – Layout of Site 4 (Sprinkler Irrigation)



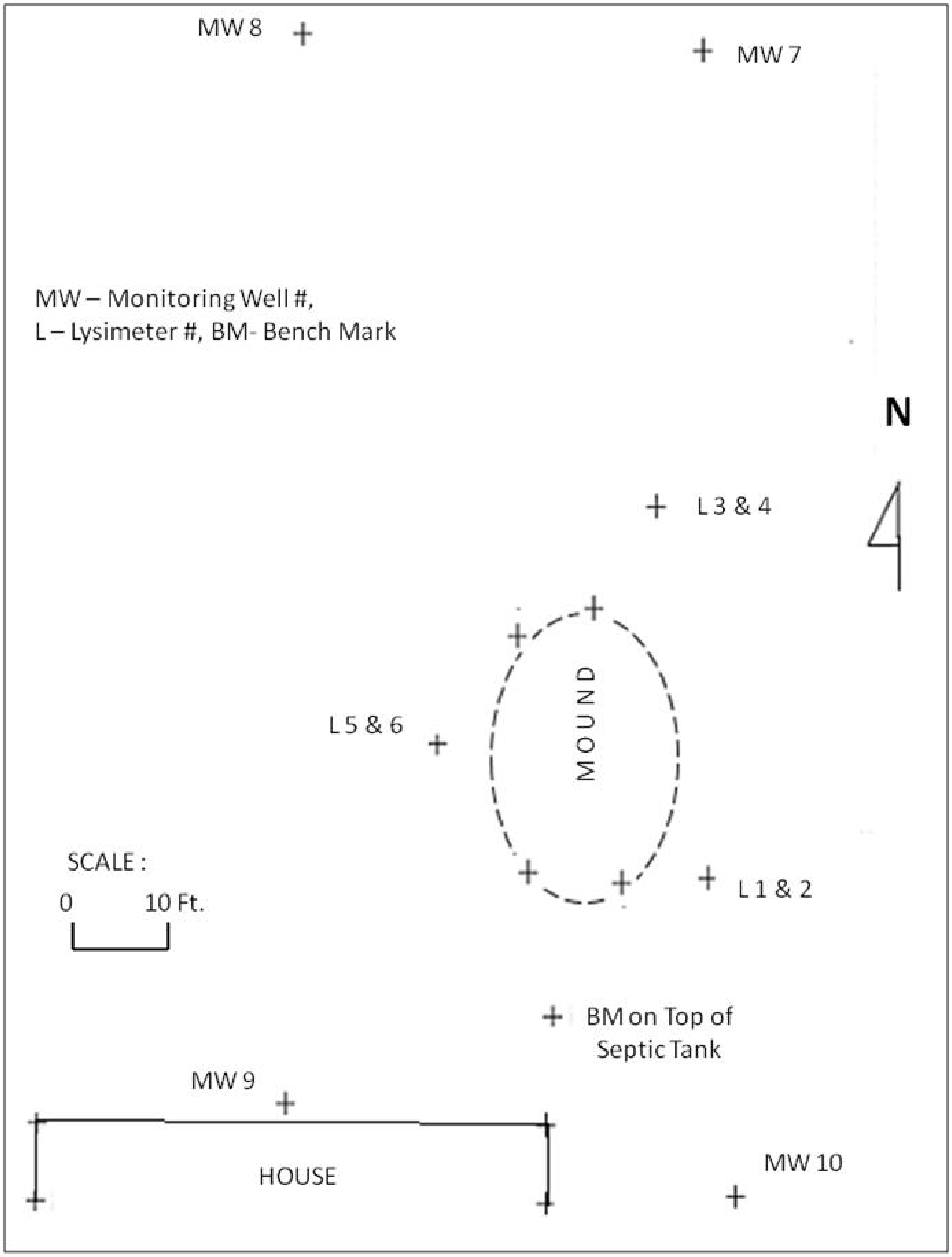
Site 4 : Sprinkler Irrigation Site
Address : 3213 Westlane, Gautier, MS 39553

Appendix F.5 – Layout of Site 5 (Sand Mound)



Site 5: Sand Mound Site
Address : 15486 Mark West Road, Gulfport, MS 39503

Appendix F.6 – Layout of Site 6 (Sand Mound)



Site 6: Sand Mound Site
Address : 7096 Woolmarket, Biloxi, MS 39532

APPENDIX G

TYPICAL VALUES, RANGES, AND DISTRIBUTION OF KINETIC AND STOICHIOMETRIC PARAMETERS

APPENDIX G

Typical Values, Ranges, and Distribution of Kinetic and Stoichiometric Parameters at Neutral pH and 20^{o} C for Domestic Wastewater

Symbol	Units	Statistical Parameters		Mean value
		ξ	σ	
Heterotrophic coefficients				
Y_H	mg biomass COD formed/ mg COD oxidized	-0.45	0.12	0.64
μ_H	day^{-1}	1.14	0.60	3.13
K_S	mg COD/L	1.44	0.76	4.22
b_H	day^{-1}	-1.06	0.81	0.35
K_{NO}	mg NO_3^--N/L	-1.55	1.01	0.21
$K_{O,H}$	mg O_2/L	-1.46	0.83	0.23
η_g	Fraction	[a]		0.50
Autotrophic coefficients				
Y_A	mg biomass COD formed/ mg N oxidized	-1.52	0.55	0.22
μ_A	day^{-1}	-0.51	0.44	0.60
b_A	day^{-1}	-1.97	0.28	0.14
K_{NH}	mg NH_3-N/L	-0.68	1.00	0.51
$K_{O,A}$	mg O_2/L	-0.82	0.96	0.44
Hydrolysis coefficients				
k_h	mg slowly biodegradable COD/ mg cell COD-day	0.83	0.36	2.29
K_X	mg slowly biodegradable COD/ mg cell COD	-2.82	1.34	0.06
η_h	Fraction	-0.86	0.62	0.42
Other coefficients				
f'_D	mg debris COD/ mg biomass COD	[b]		0.08
$i_{N/XB}$	mg N/ mg COD in active biomass	[b]		0.086
$i_{N/XD}$	mg N/ mg COD in biomass debris	[b]		0.06
k_a	L/ mg biomass COD - hour	[b]		0.1608

Source: Cox, 2004

[a] η_g follows a uniform PDF within the range of 0.10 to 0.90

[b] Values represent recommended parameter values

APPENDIX H

FIELD SAMPLING TEST RESULTS FOR PROJECT SITES

Notes for the Tables

a – Insufficient sample

bdl – Below detection limit

d – Lysimeter nos. 1, 3, 5 at shallow level, nos. 2, 4, 6 at deep levels

e – Wells at 6 ft. depth at all four locations

ex – Experimental error

– below accuracy limit

Appendix H.1 – Field Sampling Test Results for Site No 1

Sample		**Lysimeter Samples***					**Monitoring Well Samples****				
		1	2	3	4	5	6	7	8	9	10
Trip	**Parameter**	**COD, mg/L**									
1	Date: 11.01.2008	a	a	a	a	a	a	a	a	a	a
3	Date: 11.30.2008	55.40	33.90	53.88	20.80	45.98	9.93	a	a	a	a
5	Date: 12.14.2008	41.50	6.30	30.98	29.05	68.88	17.70	a	a	a	a
7	Date: 12.30.2008	18.75	7.33	30.98	22.93	67.90	4.10	a	a	a	a
9	Date: 01.16.2009	26.00	9.93	18.75	20.80	4.10	3.08	a	a	a	a
11	Date: 02.05.2009	41.50	21.90	41.50	23.95	82.70	21.90	a	a	a	a
13	Date: 03.02.2009	89.78	32.00	53.88	43.43	68.10	43.43	73.98	a	a	a
15	Date: 03.19.2009	58.28	8.80	27.00	24.98	48.60	22.93	33.03	85.40	a	21.90
17	Date: 04.07.2009	8.80	17.70	10.95	13.30	22.93	11.98	23.95	28.03	14.40	8.80
19	Date: 04.23.2009	a	3.08	a	17.70	a	10.95	11.98	22.93	15.43	53.88
21	Date: 05.10.2009	a	a	a	a	a	a	60.53	a	a	a
	Parameter	**BOD, mg/L**									
1	Date: 11.01.2008	a	a	a	a	a	a	a	a	a	
3	Date: 11.30.2008	a	a	a	a	a	a	a	a	a	
5	Date: 12.14.2008	a	a	a	a	a	a	a	a	a	
7	Date: 12.30.2008	a	a	a	a	a	a	a	a	a	
9	Date: 01.16.2009	a	a	a	a	a	a	a	a	a	
11	Date: 02.05.2009	a	a	a	a	a	a	a	a	a	
13	Date: 03.02.2009	60.00	10.24	a	10.56	13.50	9.90	a	a	a	a
15	Date: 03.19.2009	a	a	a	4.73	a	4.08	5.22	5.67	a	a
17	Date: 04.07.2009	6.42	2.35	5.65	3.80	9.18	3.60	5.31	2.03	1.60	1.20
19	Date: 04.23.2009	a	a	a	a	a	a	a	a	a	a
21	Date: 05.10.2009	a	11.37	38.21	4.27	a	10.97	13.24	5.77	8.33	34.57
	Parameter	**TKN, mg/L**									
1	Date: 11.01.2008	a	a	a	a	a	a	a	a	a	a
3	Date: 11.30.2008	0.54	0.36	0.38	0.33	0.50	0.29	a	a	a	a
5	Date: 12.14.2008	3.08	0.66	1.68	0.57	5.16	0.25	a	a	a	a
7	Date: 12.30.2008	0.92	0.58	1.73	0.35	0.82	0.09	a	a	a	a
9	Date: 01.16.2009	0.68	0.44	1.60	0.50	0.59	0.95	a	a	a	a
11	Date: 02.05.2009	1.43	0.67	1.34	0.88	3.12	0.62	a	a	a	a
13	Date: 03.02.2009	a	a	a	a	a	a	0.80	a	a	a
15	Date: 03.19.2009	a	0.78	1.16	0.45	1.84	0.17	0.28	0.69	a	0.29
17	Date: 04.07.2009	0.26	0.21	0.32	0.23	a	0.30	0.32	0.37	0.28	0.31
19	Date: 04.23.2009	a	0.36	a	0.41	a	0.16	0.35	0.32	0.32	0.83
21	Date: 05.10.2009	a	a	a	a	a	0.81	0.55	a	a	a
	Parameter	**NH_3-N, mg/L**									
1	Date: 11.01.2008	a	a	a	a	a	a	a	a	a	a
3	Date: 11.30.2008	a	0.29	a	0.23	a	0.19	a	a	a	a
5	Date: 12.14.2008	0.58	0.24	1.40	0.06	1.62	0.05	a	a	a	a
7	Date: 12.30.2008	0.22	0.00	0.25	0.00	0.52	0.00	a	a	a	a
9	Date: 01.16.2009	0.07	0.03	0.11	0.00	0.03	0.05	a	a	a	a
11	Date: 02.05.2009	0.24	0.06	0.14	0.03	0.33	0.04	a	a	a	a
13	Date: 03.02.2009	a	a	a	a	a	a	0.02	a	a	a
15	Date: 03.19.2009	a	0.06	a	0.00	0.49	0.00	0.00	0.00	a	0.00
17	Date: 04.07.2009	0.03	0.02	0.02	0.02	a	0.00	0.00	0.00	0.00	0.00
19	Date: 04.23.2009	a	0.03	a	0.00	a	0.00	0.00	0.03	0.00	0.02
21	Date: 05.10.2009	a	a	a	a	a	0.03	0.01	a	a	a
	Parameter	**FC, cfu/100 ml**									
1	Date: 11.01.2008	a	a	a	a	a	a	a	a	a	a
3	Date: 11.30.2008	a	a	a	a	a	a	a	a	a	a
5	Date: 12.14.2008	a	a	a	a	a	a	a	a	a	a
7	Date: 12.30.2008	a	a	a	a	a	a	a	a	a	a
9	Date: 01.16.2009	a	a	a	a	a	a	a	a	a	a
11	Date: 02.05.2009	a	a	a	a	a	a	a	a	a	a
13	Date: 03.02.2009	0	a	0	16	0	12	a	a	a	a
15	Date: 03.19.2009	a	a	a	a	a	a	a	a	a	a
17	Date: 04.07.2009	a	a	a	a	a	a	a	a	a	a
19	Date: 04.23.2009	0	14	0	1	0	0	2	7	3	0
21	Date: 05.10.2009	16	104	0	0	0	0	28	12	8	a

Appendix H.2 – Field Sampling Test Results for Site No 2

Sample		Lysimeter Samples*					Monitoring Well Samples**				
		1	2	3	4	5	6	7	8	9	10
Trip	**Parameter**	**COD, mg/L**									
1	Date: 11.02.2008	a	a	28.03	61.55	136.90	95.90	129.50	148.20	82.65	6.30
3	Date: 11.29.2008	181.60	a	76.88	71.20	86.85	179.55	105.70	165.30	157.50	2.05
5	Date: 12.14.2008	158.93	297.50	61.95	41.60	58.10	114.50	170.85	143.85	197.15	32.90
7	Date: 12.30.2008	145.90	230.70	73.80	35.40	79.35	90.20	81.63	77.00	105.85	5.28
9	Date: 01.16.2009	198.00	216.50	117.40	26.60	79.35	79.35	163.40	135.00	85.40	17.70
11	Date: 02.05.2009	256.10	225.00	86.85	30.85	67.80	67.80	144.40	130.20	56.23	8.20
13	Date: 03.02.2009	56.05	90.20	75.40	45.85	79.35	75.40	247.75	94.00	270.00	4.10
15	Date: 03.19.2009	74.10	112.90	44.45	27.00	58.70	71.93	103.20	112.90	163.80	29.05
17	Date: 04.07.2009	39.68	58.70	26.00	22.93	85.43	41.50	143.60	97.55	134.05	19.78
19	Date: 04.23.2009	85.40	41.50	59.50	86.08	71.20	82.65	98.40	237.10	274.60	14.40
21	Date: 05.10.2009	66.88	58.28	52.00	134.05	83.38	91.35	156.65	98.90	137.75	0.00
	Parameter	**BOD, mg/L**									
1	Date: 11.02.2008	a	a	a	a	10.01	a	8.00	9.05	9.29	4.15
3	Date: 11.29.2008	a	a	a	a	a	a	8.16	2.58	8.82	1.58
5	Date: 12.14.2008	a	a	a	a	2.04	2.16	0.54	1.29	0.66	0.72
7	Date: 12.30.2008	a	a	a	0.42	1.98	2.82	bdl	3.42	29.22	1.92
9	Date: 01.16.2009	a	a	a	36.21	18.45	12.39	17.31	15.21	12.57	17.43
11	Date: 02.05.2009	a	a	a	4.85	8.02	4.71	3.15	6.45	0.99	2.49
13	Date: 03.02.2009	a	a	6.00	6.96	8.16	9.48	3.90	3.39	2.31	2.52
15	Date: 03.19.2009	9.53	23.90	6.42	8.46	11.10	9.42	4.71	6.84	3.78	2.55
17	Date: 04.07.2009	6.70	25.98	8.90	4.25	3.75	3.65	3.72	3.27	2.25	0.97
19	Date: 04.23.2009	a	a	a	a	a	a	a	a	a	a
21	Date: 05.10.2009	a	a	a	a	21.13	24.97	4.03	4.12	7.60	0.00
	Parameter	**TKN, mg/L**									
1	Date: 11.02.2008	a	a	a	2.12	5.54	5.30	1.74	6.38	7.80	0.40
3	Date: 11.29.2008	0.54	0.36	0.38	0.33	0.50	0.29	0.85	1.47	1.13	0.26
5	Date: 12.14.2008	2.71	4.39	1.70	0.62	0.87	1.74	2.64	1.75	4.10	0.18
7	Date: 12.30.2008	6.70	5.11	1.11	0.41	0.79	1.21	0.62	2.25	3.04	0.16
9	Date: 01.16.2009	5.06	4.79	1.04	0.20	0.66	0.89	2.75	1.40	2.55	0.10
11	Date: 02.05.2009	10.84	4.10	1.43	0.34	1.02	1.00	1.22	1.51	0.58	0.18
13	Date: 03.02.2009	1.22	2.09	1.10	0.49	0.94	0.76	1.85	1.19	2.56	0.14
15	Date: 03.19.2009	2.63	3.08	0.76	0.38	1.03	0.59	2.83	2.09	5.67	0.25
17	Date: 04.07.2009	0.42	0.81	0.58	0.39	0.99	0.70	2.31	1.14	2.20	0.13
19	Date: 04.23.2009	2.73	1.82	1.03	1.55	2.06	1.45	1.05	2.02	2.64	0.17
21	Date: 05.10.2009	4.81	3.24	1.74	2.20	3.31	2.53	1.37	1.43	1.52	0.29
	Parameter	**NH_3-N, mg/L**									
1	Date: 11.02.2008	a	a	1.03	a	0.28	0.17	0.10	0.56	0.12	0.06
3	Date: 11.29.2008	a	a	0.31	0.31	0.42	a	0.03	0.00	0.03	0.04
5	Date: 12.14.2008	2.38	1.49	0.13	0.02	0.00	0.03	0.05	0.02	0.05	0.00
7	Date: 12.30.2008	0.12	0.08	0.05	0.00	0.00	0.05	0.03	0.01	0.04	0.01
9	Date: 01.16.2009	0.09	0.09	0.05	0.02	0.00	0.02	0.03	0.00	0.04	0.00
11	Date: 02.05.2009	0.18	0.28	0.17	0.00	0.00	0.03	0.04	0.00	0.03	0.00
13	Date: 03.02.2009	0.05	0.02	0.02	0.00	0.00	0.00	0.00	0.00	0.03	0.00
15	Date: 03.19.2009	0.02	0.02	0.02	0.00	0.00	0.00	0.02	0.00	0.03	0.00
17	Date: 04.07.2009	0.02	0.02	0.01	0.02	0.02	0.06	0.03	0.02	0.03	0.02
19	Date: 04.23.2009	0.04	0.02	0.00	0.00	0.21	0.05	0.00	0.02	0.02	0.00
21	Date: 05.10.2009	0.21	0.09	0.04	0.03	1.15	0.11	0.06	0.02	0.04	0.01
	Parameter	**FC, cfu/100 ml**									
1	Date: 11.02.2008	a	a	a	a	a	a	a	a	a	a
3	Date: 11.29.2008	a	a	a	a	a	a	a	a	a	a
5	Date: 12.14.2008	a	a	a	a	12	a	a	a	11	6
7	Date: 12.30.2008	a	a	a	25	0	0	0	0	5	17
9	Date: 01.16.2009	a	a	a	0	51	0	4	8	2	0
11	Date: 02.05.2009	a	a	a	0	0	1	8	17	0	0
13	Date: 03.02.2009	0	4	0	20	10	0	16	0	4	1
15	Date: 03.19.2009	a	a	8	4	0	34	8	80	0	14
17	Date: 04.07.2009	a	0	3	0	1	7	0	6	0	0
19	Date: 04.23.2009	a	a	0	0	0	0	6	0	0	3
21	Date: 05.10.2009	a	a	a	a	10	6	0	0	0	0

Appendix H.3 – Field Sampling Test Results for Site No 3

Sample		Lysimeter Samples*					Monitoring Well Samples**				
		1	2	3	4	5	6	7	8	9	10
Trip	**Parameter**	**COD, mg/L**									
2	Date: 11.23.2008	21.90	a	a	a	58.70	a	54.90	26.00	33.90	a
4	Date: 12.07.2008	a	a	a	a	a	a	a	a	a	a
6	Date: 12.21.2008	485.09	83.00	293.90	52.00	443.96	114.50	17.70	48.60	30.00	5.28
8	Date: 01.07.2009	434.25	67.80	137.75	39.55	668.75	90.20	17.70	40.70	13.30	35.88
10	Date: 01.26.2009	416.88	12.60	323.60	30.85	778.13	37.50	1.03	33.03	33.90	28.03
12	Date: 02.16.2009	453.50	81.40	559.38	30.85	665.13	64.00	15.43	30.98	24.98	48.03
14	Date: 03.09.2009	440.50	84.80	435.50	43.80	492.00	86.85	15.43	39.68	14.40	30.00
16	Date: 03.29.2009	379.25	64.00	334.38	37.50	370.50	32.90	14.40	26.00	17.70	45.10
18	Date: 04.14.2009	334.38	30.00	307.75	2.05	109.50	13.30	0.00	5.28	2.05	2.05
20	Date: 05.01.2009	340.50	77.00	379.25	23.95	140.13	32.00	13.30	32.00	18.75	37.70
	Parameter	**BOD, mg/L**									
2	Date: 11.23.2008										
4	Date: 12.07.2008										
6	Date: 12.21.2008		0.48		6.00			0.21	0.83	0.03	1.08
8	Date: 01.07.2009	15.53	1.14	65.50	bdl		20.13	1.56	0.21	11.48	
10	Date: 01.26.2009		3.48		15.54						
12	Date: 02.16.2009	33.94	5.96	26.44	7.61		4.39	2.45	2.84	4.58	0.16
14	Date: 03.09.2009		3.07		2.63		2.33	0.09			
16	Date: 03.29.2009	5.10	2.33	7.28	1.35	4.65		0.45		0.57	0.72
18	Date: 04.14.2009	13.20	7.70	14.60	1.87	11.35	2.05	0.00	0.93	1.73	1.84
20	Date: 05.01.2009				3.47		4.77	0.62	1.99	0.17	0.92
	Parameter	**TKN, mg/L**									
2	Date: 11.23.2008	25.20				1.29		0.04	13.04	0.09	
4	Date: 12.07.2008										
6	Date: 12.21.2008	2.40	1.05	15.86	0.55	1.10	0.57	0.16	0.16	0.33	0.11
8	Date: 01.07.2009	20.94	1.50	26.77	0.44	3.17	0.51	0.10	0.18	0.17	0.09
10	Date: 01.26.2009	7.30	0.93	14.19	0.50	18.83	0.59	0.27	0.32	0.35	0.24
12	Date: 02.16.2009	17.26	1.42	8.04	0.23	13.77	0.48	0.14	0.26	0.36	0.28
14	Date: 03.09.2009	7.79	0.34	10.92	0.42	7.42	0.49	0.19	0.27	0.28	0.17
16	Date: 03.29.2009	9.74	1.28	8.58	0.71	6.88	0.50	0.39	0.42	0.42	0.55
18	Date: 04.14.2009	4.90	1.01	7.23	0.41	2.42	0.32	0.15	0.14	0.13	0.08
20	Date: 05.01.2009	3.54	1.00	5.49	0.26	1.54	0.28	0.15	0.17	0.07	0.09
	Parameter	**NH_3-N, mg/L**									
2	Date: 11.23.2008	1.97						0.37	0.20	0.23	
4	Date: 12.07.2008	0.34		0.04	0.05						
6	Date: 12.21.2008	0.09	0.00	0.05	0.00	0.00	0.00	0.00	0.00	0.04	0.00
8	Date: 01.07.2009	0.55	0.02	0.03	0.00	0.06	0.00	0.00	0.00	0.00	0.00
10	Date: 01.26.2009	0.16	0.04	0.04	0.00	0.09	0.00	0.00	0.00	0.00	0.00
12	Date: 02.16.2009	0.24	0.01	0.03	0.00	0.02	0.00	0.00	0.00	0.02	0.00
14	Date: 03.09.2009	0.09	0.03	0.04	0.02	0.04	0.02	0.02	0.00	0.00	0.02
16	Date: 03.29.2009	0.28	0.02	0.02	0.00	0.03	0.00	0.00	0.00	0.00	0.00
18	Date: 04.14.2009	0.58	0.03	0.02	0.01	0.05	0.01	0.01	0.01	0.01	0.01
20	Date: 05.01.2009	0.27	0.13	0.05	0.00	0.06	0.00	0.00	0.00	0.00	0.00
	Parameter	**FC, cfu/100 ml**									
2	Date: 11.23.2008										
4	Date: 12.07.2008										
6	Date: 12.21.2008							6	3	1	18
8	Date: 01.07.2009	14	4	20	97		7	8	5	46	16
10	Date: 01.26.2009		4		84	0	0	0	0	1	4
12	Date: 02.16.2009	0	0		9		0	0	0	5	0
14	Date: 03.09.2009		2		36		7	44	1	11	4
16	Date: 03.29.2009	1	0	0	9	5	0	2	21	10	50
18	Date: 04.14.2009	2.00	0	2	0	0	0	3	7	4	0
20	Date: 05.01.2009				6		0	0	1	0	0

Appendix H.4 – Field Sampling Test Results for Site No 4

Sample		Lysimeter Samples*					Monitoring Well Samples**				
		1	2	3	4	5	6	7	8	9	10
Trip	**Parameter**	**COD, mg/L**									
1	Date: 11.02.2008							24.98	20.80	10.95	7.33
2	Date: 11.30.2008	165.00	223.75	54.90	165.00	7.33	55.40	2.05	21.90	13.30	22.93
3	Date: 12.13.2008	151.70	69.70	67.80	172.15	35.40	61.95	26.60	6.15	21.90	17.60
4	Date: 12.29.2009	49.95	32.90	49.95	43.80	12.60	39.55	18.75	1.03	18.75	7.33
5	Date: 01.15.2009	96.05	17.60	43.80	45.85	23.95	56.05	23.95	3.08	10.95	8.80
6	Date: 02.04.2009	86.85	26.60	45.85	60.00	64.00	60.00	27.00	21.90	9.93	10.95
7	Date: 03.01.2009	56.05	8.20	35.40	32.90	23.95	23.95	19.78	10.95	7.33	6.30
8	Date: 03.18.2009	89.78	4.10	19.78	30.00	16.45	13.30	2.05	2.05	19.78	11.98
9	Date: 04.06.2009	106.20	30.00	49.45	42.40	53.88	34.85	29.05	24.98	30.00	26.00
10	Date: 04.22.2009	87.10	6.30	52.00	42.40	20.80	27.00	19.78	13.30	10.95	18.75
11	Date: 05.09.2009		35.88		58.70			13.30	10.95	8.80	10.95
	Parameter	**BOD, mg/L**									
1	Date: 11.02.2008							9.14	9.05	9.20	4.92
2	Date: 11.30.2008	6.96						5.46	3.96	1.80	3.36
3	Date: 12.13.2008	8.78	10.70	8.60	10.75	5.79	6.36	5.61	4.89	5.28	5.76
4	Date: 12.29.2009	4.56	1.38	33.00	2.46	3.24	33.60	1.20	0.96	1.05	2.10
5	Date: 01.15.2009	26.67	11.37	21.75	17.79	18.21	40.41	17.68	2.62	10.28	5.15
6	Date: 02.04.2009	3.87	0.99	2.35	2.55	2.73	3.99	4.41	1.65	1.29	3.69
7	Date: 03.01.2009	42.93	3.96	23.72	5.46	6.42	4.92	3.66	0.87	1.62	1.98
8	Date: 03.18.2009	9.53	3.45	12.38	17.33	2.58	3.30	1.44	1.38	3.06	2.49
9	Date: 04.06.2009	3.06	2.33	4.75	0.30	2.93	0.64			0.39	0.87
10	Date: 04.22.2009										
11	Date: 05.09.2009							0.22	2.88	6.14	4.19
	Parameter	**TKN, mg/L**									
1	Date: 11.02.2008							0.15	0.93	0.48	0.15
2	Date: 11.30.2008	4.08	4.37	1.22	10.73	0.42	0.27	0.11	0.10	0.08	0.09
3	Date: 12.13.2008	2.61	3.61	1.20	4.07	0.28	0.16	0.17	0.05	0.06	0.07
4	Date: 12.29.2009	1.35	0.41	0.90	1.54	0.14	0.22	0.14	0.03	0.11	0.05
5	Date: 01.15.2009	1.45	0.43	0.66	0.86	0.53	0.47	0.25	0.08	0.13	0.12
6	Date: 02.04.2009	1.34	0.30	0.75	0.66	0.74	0.50	0.15	0.07	0.09	0.07
7	Date: 03.01.2009	0.84	0.18	1.06	0.44	0.63	0.12	0.07	0.03	0.04	0.03
8	Date: 03.18.2009	2.13	0.40	1.36	0.79	0.36	0.42	0.23	0.23	0.27	0.21
9	Date: 04.06.2009	1.29	0.24	0.40	0.32	0.35	0.16	0.12	0.04	0.15	0.18
10	Date: 04.22.2009	2.48	1.27	1.62	1.23	1.52	1.06	0.27	0.05	0.10	0.17
11	Date: 05.09.2009		1.45		1.43			0.17	0.12	0.10	0.18
	Parameter	**NH_3-N, mg/L**									
1	Date: 11.02.2008							0.10	0.24	0.07	0.13
2	Date: 11.30.2008	0.12	0.09	0.09	0.11	0.00	0.00	0.00	0.00	0.00	0.00
3	Date: 12.13.2008	0.33	0.09	0.04	0.33	0.02	0.06	0.06	0.05	0.06	0.07
4	Date: 12.29.2009	0.16	0.00	0.00	0.14	0.00	0.00	0.00	0.00	0.00	0.00
5	Date: 01.15.2009	0.04	0.03	0.03	0.00	0.03	0.00	0.00	0.00	0.00	0.00
6	Date: 02.04.2009	0.04	0.00	0.05	0.00	0.00	0.00	0.00	0.00	0.00	0.00
7	Date: 03.01.2009	0.09	0.03	0.03	0.00	0.00	0.00	0.00	0.00	0.03	0.00
8	Date: 03.18.2009	0.07	0.01	0.02	0.04	0.02	0.02	0.00	0.00	0.00	0.00
9	Date: 04.06.2009	0.11	0.04	0.03	0.15	0.04	0.08	0.00	0.00	0.00	0.00
10	Date: 04.22.2009	0.08	0.02		0.07	0.07	0.03	0.00	0.00	0.00	0.00
11	Date: 05.09.2009							0.01	0.01	0.01	0.01
	Parameter	**FC, cfu/100 ml**									
1	Date: 11.02.2008										
2	Date: 11.30.2008										
3	Date: 12.13.2008	25		6		6		17	20	12	35
4	Date: 12.29.2009	5	0	5	0	8	5		31	3	19
5	Date: 01.15.2009	2	1	10	18	28	2	36	0	2	8
6	Date: 02.04.2009		0	0	6	0	0	23	0	1	2
7	Date: 03.01.2009	24	4	0	0	2	0	12	58	58	74
8	Date: 03.18.2009		0	4	4	6	0	26	4	0	0
9	Date: 04.06.2009	4	2	2	1	6	5		7	44	26
10	Date: 04.22.2009							68	14	4	9
11	Date: 05.09.2009										

Appendix H.5 – Field Sampling Test Results for Site No 5

Sample		Lysimeter Samples*					Monitoring Well Samples**				
		1	2	3	4	5	6	7	8	9	10
Trip	**Parameter**	**COD, mg/L**									
2	Date: 11.23.2008	13.30	13.30	22.93	6.30	35.88	14.40	9.93	13.30	11.98	
4	Date: 12.06.2008	21.90	2.05	13.30	2.05	11.98	14.40	6.30	8.80	13.30	
6	Date: 12.22.2008	8.80	15.43	18.75	6.30	11.98	16.45	4.10	6.30	6.30	
8	Date: 01.08.2009	6.30	8.80	1.03	3.08	14.40	6.30	4.10	8.80	7.33	
10	Date: 01.27.2009	1.03	15.43	10.95	10.95	11.98	3.08	13.30	7.33	5.28	1.03
12	Date: 02.17.2009	13.30	17.70	17.70	13.30	17.70	13.30	4.10	10.95	7.33	4.10
14	Date: 03.10.2009	8.80	13.30	11.98	13.30	18.75	14.40	7.33	5.28	8.80	23.95
16	Date: 03.30.2009	6.30	10.95	15.43	9.93	14.40	1.03	4.10	5.28	6.30	7.33
18	Date: 04.15.2009	4.10	27.00	9.93	6.30	8.80	9.93	2.05	4.10	6.30	10.95
20	Date: 05.02.2009	6.30	34.85	13.30	11.98	17.70	16.45	4.10	8.80	8.80	27.00
	Parameter	**BOD, mg/L**									
2	Date: 11.23.2008							5.82	6.00		
4	Date: 12.06.2008							2.31	1.83		
6	Date: 12.22.2008	1.14	0.12		0.12	0.60	0.78	0.21	0.84	0.36	
8	Date: 01.08.2009					1.88	0.93		3.18		
10	Date: 01.27.2009		2.64	0.13		1.91	2.29		0.57	1.08	2.21
12	Date: 02.17.2009	4.46	5.15	7.48	5.96	3.96	5.09	3.65	6.04	3.28	3.02
14	Date: 03.10.2009	0.75	3.33	a	2.60	1.86	0.72		1.35		
16	Date: 03.30.2009	1.38	1.92	3.84	0.36	1.20	0.51		1.29	0.12	
18	Date: 04.15.2009	3.30	8.34	7.64	5.90	6.39	8.25	1.80	1.40	1.20	1.50
20	Date: 05.02.2009							0.74	0.17	2.06	15.47
	Parameter	**TKN, mg/L**									
2	Date: 11.23.2008	7.36	3.24	5.79	10.02	1.95	1.00	1.88	1.68	2.41	
4	Date: 12.06.2008	1.74	0.52	0.67	0.60	1.19	0.58	0.08	0.17	0.23	
6	Date: 12.22.2008	0.57	0.28	0.30	0.16	0.46	0.31	0.07	0.12	0.12	
8	Date: 01.08.2009	0.66	0.63	0.86	0.53	0.31	0.43	0.23	0.28	0.44	
10	Date: 01.27.2009	0.51	0.28	0.42	0.16	0.41	0.22	0.08	0.10	0.20	0.08
12	Date: 02.17.2009	0.35	0.38	0.35	0.21	0.20	0.17	0.06	0.06	0.10	0.08
14	Date: 03.10.2009	0.22	0.15	0.34	0.33	7.99	0.25	0.07	0.11	0.28	0.22
16	Date: 03.30.2009	0.36	0.70	0.43	0.23	0.48	0.34	0.19	0.10	0.08	0.05
18	Date: 04.15.2009	0.18	0.70	0.75	0.36	0.54	0.59	0.17	0.17	0.27	0.21
20	Date: 05.02.2009	0.12	0.73	0.28	0.09	0.22	0.12	0.05	0.19	0.26	0.38
	Parameter	**NH_3-N, mg/L**									
2	Date: 11.23.2008	1.01	1.03	4.38	0.20	1.24	0.78	a	0.05	0.07	a
4	Date: 12.06.2008	0.28	0.09	0.10	0.03	0.24	0.09	0.00	0.05	0.00	a
6	Date: 12.22.2008	0.10	0.08	0.03	0.04	0.17	0.04	0.00	0.05	0.00	a
8	Date: 01.08.2009	0.09	0.07	0.03	0.09	0.03	0.04	0.00	0.00	0.04	a
10	Date: 01.27.2009	0.03	0.03	0.03	0.00	0.00	0.00	0.00	0.06	0.00	0.00
12	Date: 02.17.2009	0.00	0.07	0.00	0.05	0.00	0.00	0.00	0.03	0.00	0.00
14	Date: 03.10.2009	0.03	0.06	0.03	0.00	0.00	0.00	0.03	0.04	0.05	0.03
16	Date: 03.30.2009	0.02	0.19	0.02	0.02	0.00	0.00	0.03	0.03	0.00	0.00
18	Date: 04.15.2009	0.01	0.26	0.01	0.01	0.01	0.01	0.01	0.02	0.01	0.01
20	Date: 05.02.2009	0.00	0.46	0.00	0.00	0.00	0.00	0.00	0.03	0.00	0.00
	Parameter	**FC, cfu/100 ml**									
2	Date: 11.23.2008										
4	Date: 12.06.2008										
6	Date: 12.22.2008		3		4		0	6	8	4	
8	Date: 01.08.2009	9	0	0	6	0	0	8	4	3	
10	Date: 01.27.2009	0	4	0	0	0	0	14	0	20	34
12	Date: 02.17.2009	0	0	0	3	0	3	72	7	17	9
14	Date: 03.10.2009		2		0			0	0	15	0
16	Date: 03.30.2009	0	0	10	38	0	12	0	27	6	188
18	Date: 04.15.2009	0	0	0	0	8	20	21	22	1	1
20	Date: 05.02.2009							0	0	0	0

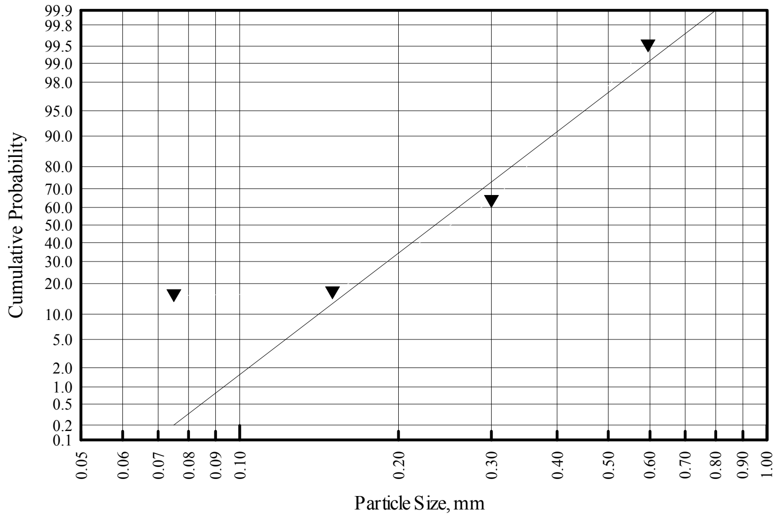
Appendix H.6 – Field Sampling Test Results for Site No 6

Sample		Lysimeter Samples*					Monitoring Well Samples**				
		1	2	3	4	5	6	7	8	9	10
Trip	**Parameter**	**COD, mg/L**									
2	Date: 12.06.2008	11.98	26.00		a	48.03	62.70	13.30	7.33	13.30	a
4	Date: 12.22.2008	68.88	24.98		7.33	93.95	19.78	71.20	14.40	63.73	89.78
6	Date: 01.08.2009	60.53	8.80	85.40	20.80	33.90	133.50	26.00	8.80	49.45	105.85
8	Date: 01.27.2009	48.60	20.80	75.85	8.80	217.15	54.90	32.00	6.30	29.05	55.40
10	Date: 02.17.2009	57.25	41.50	61.90	11.98	142.40	77.00	18.75	14.40	26.00	89.78
12	Date: 03.10.2009	65.10	29.05		20.80	133.75	71.93	10.95	10.95	28.03	88.10
14	Date: 03.30.2009	66.88	64.75		15.43	139.80	81.63	16.45	15.43	30.00	102.43
16	Date: 04.15.2009	57.25	32.00		8.80	209.65	51.50	6.30	8.80	5.28	71.93
18	Date: 05.02.2009	48.60	40.70		7.33	223.75	80.60	15.43	15.43	33.03	77.00
20											
	Parameter	**BOD, mg/L**									
2	Date: 12.06.2008		1.38					1.44		10.61	
4	Date: 12.22.2008	0.36	1.74		1.74	6.78	4.26	18.90	0.57	10.74	23.85
6	Date: 01.08.2009	1.86				3.24	0.42	5.01	0.90	11.91	16.35
8	Date: 01.27.2009	1.05	1.38		0.36	2.85	7.15	7.74	4.77	14.34	16.05
10	Date: 02.17.2009	16.17	11.25		11.91	14.61	20.31	7.70	7.97	15.83	
12	Date: 03.10.2009	2.64	0.72		1.32	6.78	5.76	3.87	0.12	5.64	6.51
14	Date: 03.30.2009	3.30	1.44		0.06	3.00	3.54	1.35	1.41	6.87	bdl
16	Date: 04.15.2009	7.91	9.27		5.61	10.23	24.99	4.02	2.31	4.80	7.35
18	Date: 05.02.2009		5.08		3.55			2.39	0.06	8.21	4.82
20											
	Parameter	**TKN, mg/L**									
2	Date: 12.06.2008	0.58	0.40			0.19					
4	Date: 12.22.2008	0.44	0.22		0.42	2.25	0.59	0.56	0.10	1.49	2.74
6	Date: 01.08.2009	1.12	0.69	2.66	0.38	3.61	0.65	0.30	0.10	2.88	2.21
8	Date: 01.27.2009	1.52	0.74	3.98	0.31	5.42	1.30	0.37	0.14	1.89	15.33
10	Date: 02.17.2009	1.67	1.09	15.64	0.51	3.38	2.06	0.24	0.16	1.37	1.51
12	Date: 03.10.2009	1.11	0.37		0.20	2.32	0.89	0.12	0.09	1.27	1.17
14	Date: 03.30.2009	1.37	0.85		0.15	2.28	0.66	0.11	0.07	1.00	1.29
16	Date: 04.15.2009	1.92	0.99		0.22	1.82	1.24	0.31	0.14	1.17	1.69
18	Date: 05.02.2009	2.63	0.97		0.43	3.23	1.73	0.38	0.17	2.58	2.08
20											
	Parameter	**NH_3-N, mg/L**									
2	Date: 12.06.2008	0.05	0.03			0.02	0.04	0.00	0.00	3.41	
4	Date: 12.22.2008	0.05	0.00		0.00	0.00	0.00	0.08	0.00	0.42	1.28
6	Date: 01.08.2009	0.00	0.04	1.38	0.06	0.10	0.07	0.03	0.03	1.30	0.71
8	Date: 01.27.2009	0.15	0.10	0.30	0.10	0.14	0.09	0.04	0.00	1.81	0.64
10	Date: 02.17.2009	0.37	0.23	0.38	0.18	0.31	0.25	0.00	0.00	1.21	1.05
12	Date: 03.10.2009	0.33	0.12		0.07	0.36	0.25	0.04	0.00	1.23	0.95
14	Date: 03.30.2009	0.48	0.28		0.09	0.49	0.35	0.03	0.02	0.92	0.54
16	Date: 04.15.2009	0.67	0.23		0.04	0.48	0.36	0.12	0.02	0.56	0.68
18	Date: 05.02.2009	0.39	0.24		0.09	0.51	0.38	0.19	0.02	2.23	1.16
20											
	Parameter	**FC, cfu/100 ml**									
2	Date: 12.06.2008										
4	Date: 12.22.2008	23	5		1	3	0	1	0	0	0
6	Date: 01.08.2009	0	0		0	7	0	248	60	227	18
8	Date: 01.27.2009	0	4		0	0	2	499	14	140	17
10	Date: 02.17.2009	0	0		0	0	6	138	0	665	17
12	Date: 03.10.2009		0		2	16	4	28	15	235	9
14	Date: 03.30.2009	0	0		4	0	2	6	0	0	0
16	Date: 04.15.2009	0	0		0	0	2	0	1	80	116
18	Date: 05.02.2009		0		0			18	6	1	14
20											

APPENDIX I

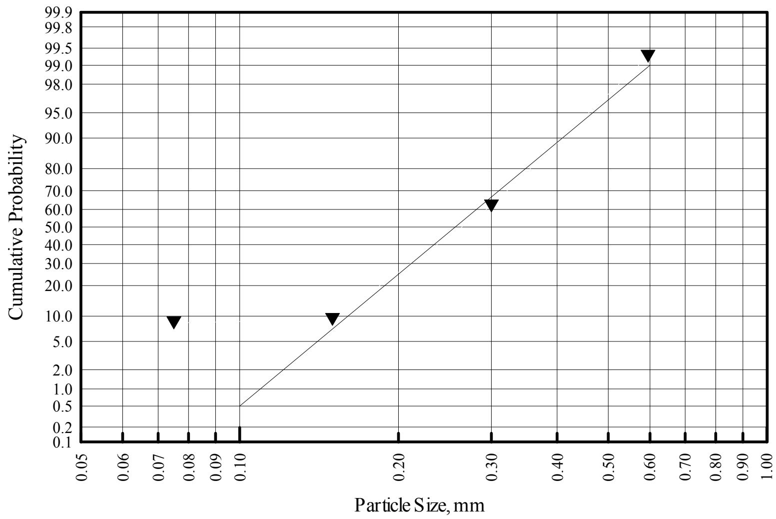
PARTICLE SIZE DISTRIBUTION CHARTS

Appendix I.1: Site 1, Monitoring Well Location #: 1 (Sample # 1/1)



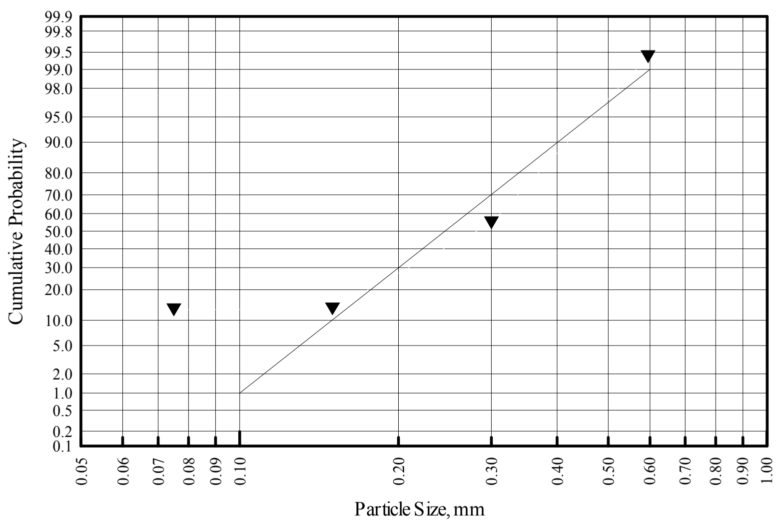
At 2 ft. depth

D_{10} = 0.14 mm,
D_{60} = 0.25 mm,
and
$C_u = D_{60}/ D_{10}$ = 1.79



At 4 ft. depth

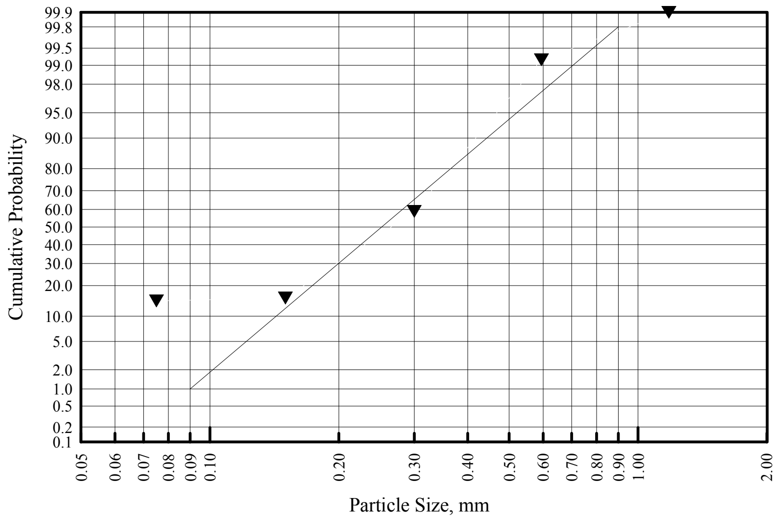
D_{10} = 0.15 mm,
D_{60} = 0.26 mm,
and
$C_u = D_{60}/ D_{10}$ = 1.73



At 6 ft. depth

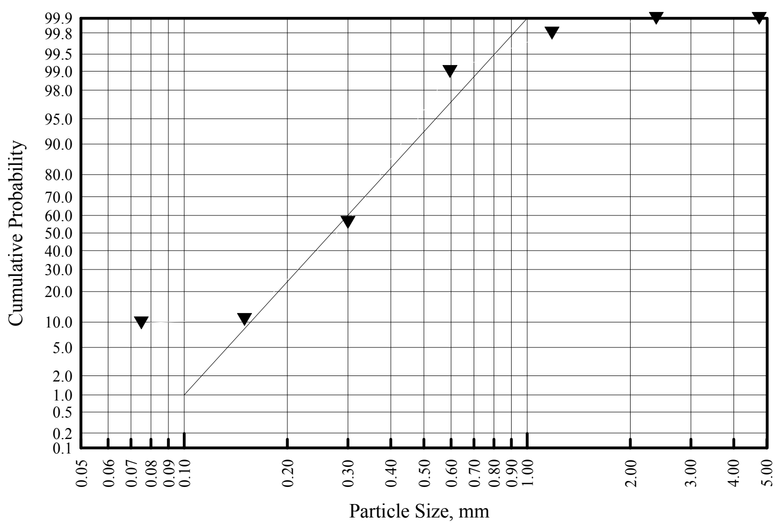
D_{10} = 0.14 mm,
D_{60} = 0.26 mm,
and
$C_u = D_{60}/ D_{10}$ = 1.86

Appendix I.1: Site 1, Monitoring Well Location #: 2 (Sample # 1/2)



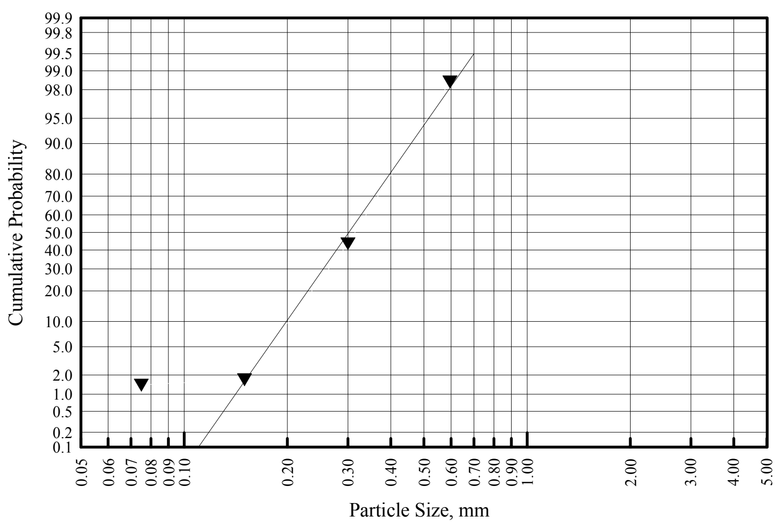
At 2 ft. depth

D_{10} = 0.14 µm,
D_{60} = 0.28 mm,
and
$C_u = D_{60}/D_{10}$ = 2.00



At 4 ft. depth

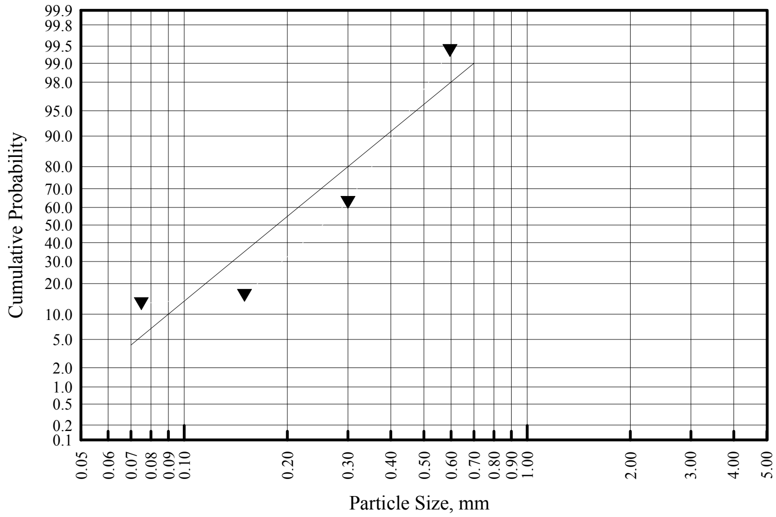
D_{10} = 0.15 µm,
D_{60} = 0.30 mm,
and
$C_u = D_{60}/D_{10}$ = 2.00



At 6 ft. depth

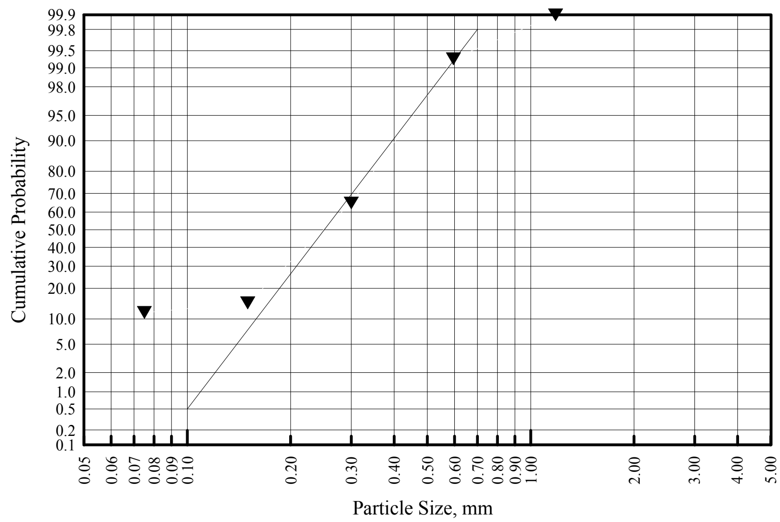
D_{10} = 0.20 µm,
D_{60} = 0.33 mm,
and
$C_u = D_{60}/D_{10}$ = 1.65

Appendix I.1: Site 1, Monitoring Well Location #: 3 (Sample # 1/3)



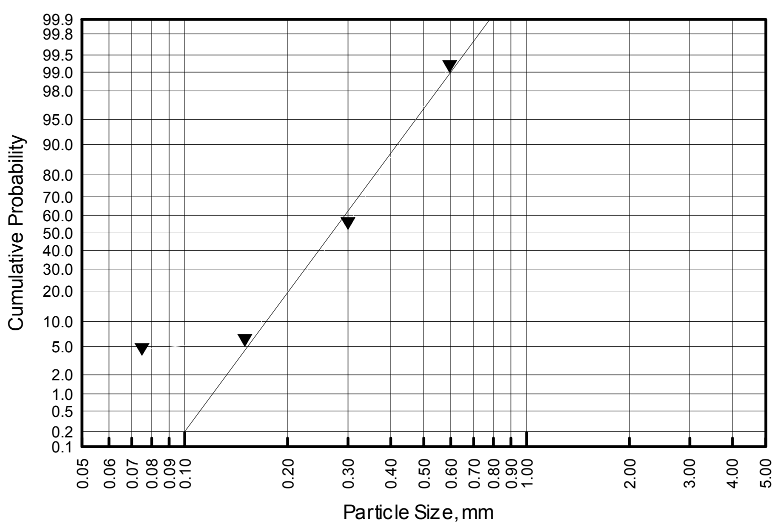
At 2 ft. depth

$D_{10} = 0.14$ mm,
$D_{60} = 0.27$ mm,
and
$C_u = D_{60}/ D_{10} = 1.92$



At 4 ft. depth

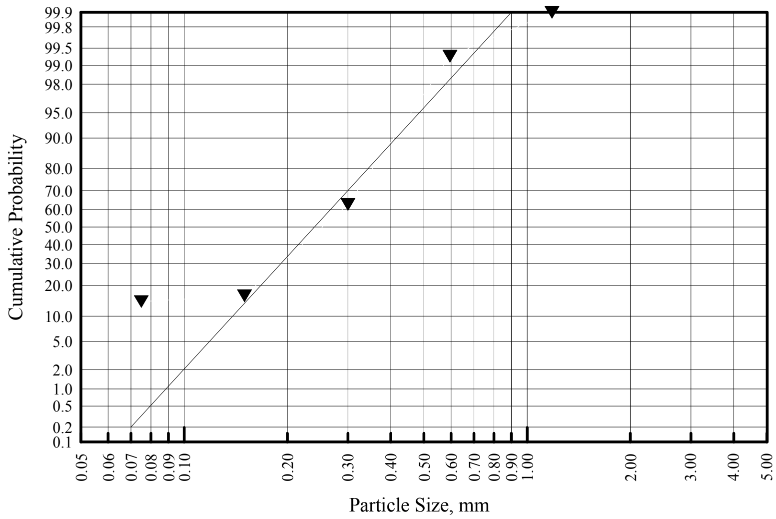
$D_{10} = 0.13$ mm,
$D_{60} = 0.26$ mm,
and
$C_u = D_{60}/ D_{10} = 2.00$



At 6 ft. depth

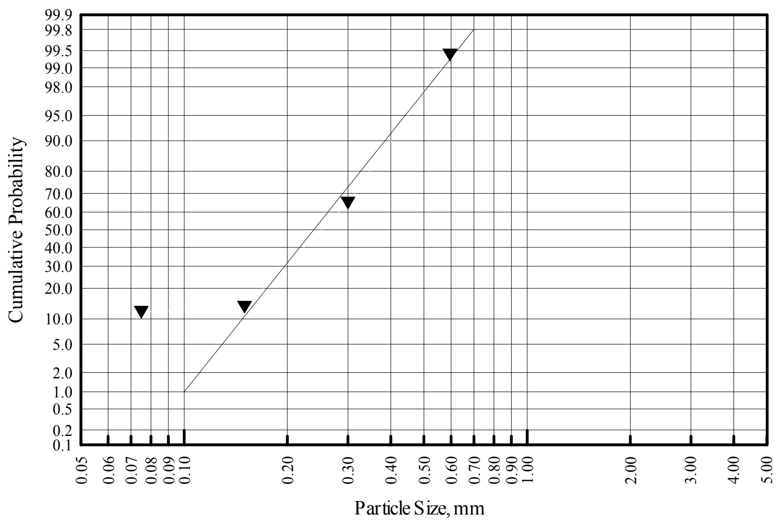
$D_{10} = 0.16$ mm,
$D_{60} = 0.30$ mm,
and
$C_u = D_{60}/ D_{10} = 1.88$

Appendix I.1: Site 1, Monitoring Well Location #: 4 (Sample # 1/4)



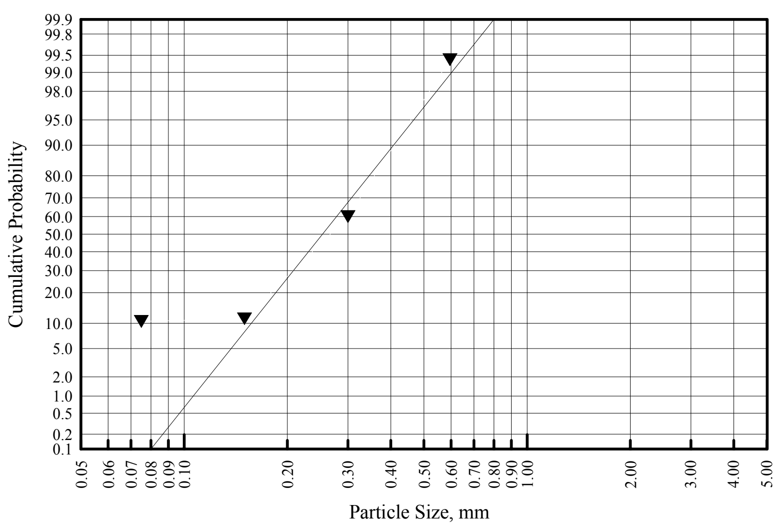
<u>At 2 ft. depth</u>

D_{10} = 0.13 mm,
D_{60} = 0.26 mm,
and
$C_u = D_{60}/ D_{10}$ = 2.00



<u>At 4 ft. depth</u>

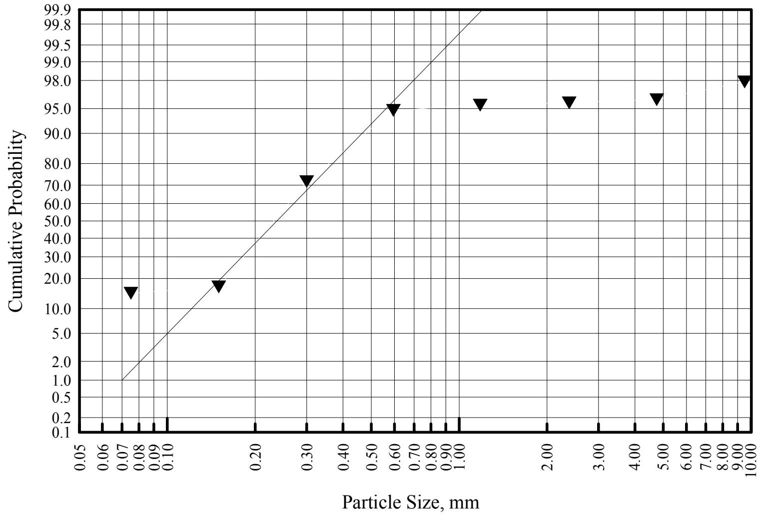
D_{10} = 0.14 mm,
D_{60} = 0.25 mm,
and
$C_u = D_{60}/ D_{10}$ = 1.79



<u>At 6 ft. depth</u>

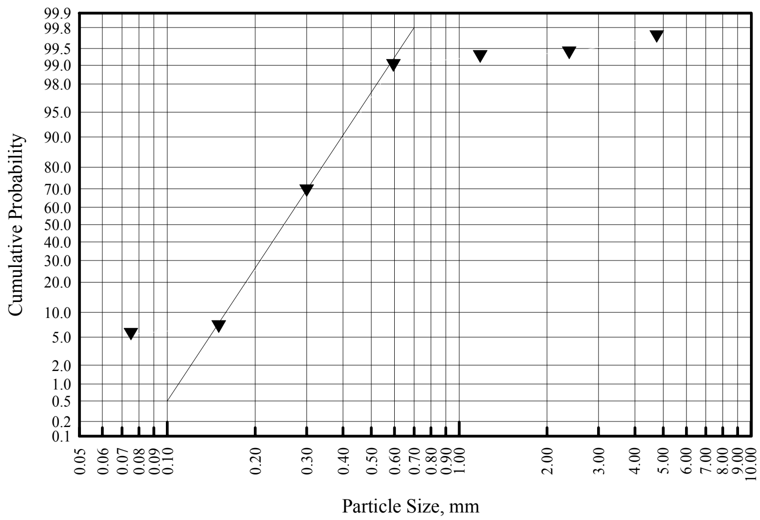
D_{10} = 0.15 mm,
D_{60} = 0.26 mm,
and
$C_u = D_{60}/ D_{10}$ = 1.73

Appendix I.2: Site 2, Monitoring Well Location #: 1 (Sample # 2/1)



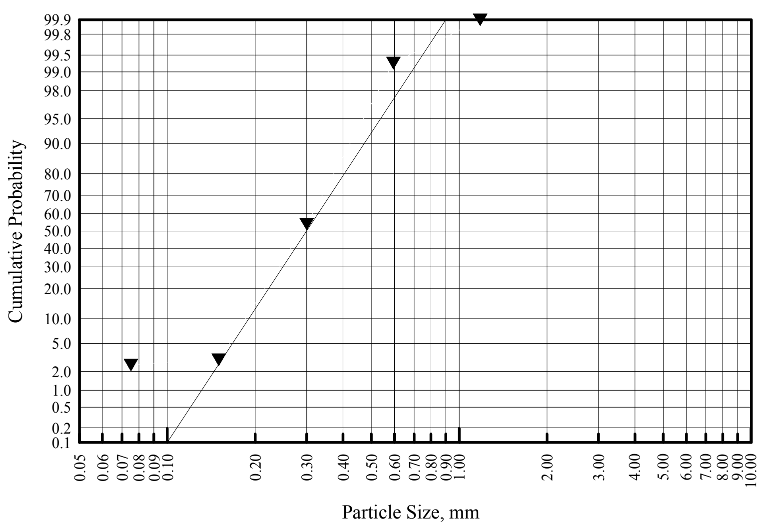
At 2 ft. depth

D_{10} = 0.12 mm,
D_{60} = 0.26 mm,
and
$C_u = D_{60}/ D_{10}$ = 2.17



At 4 ft. depth

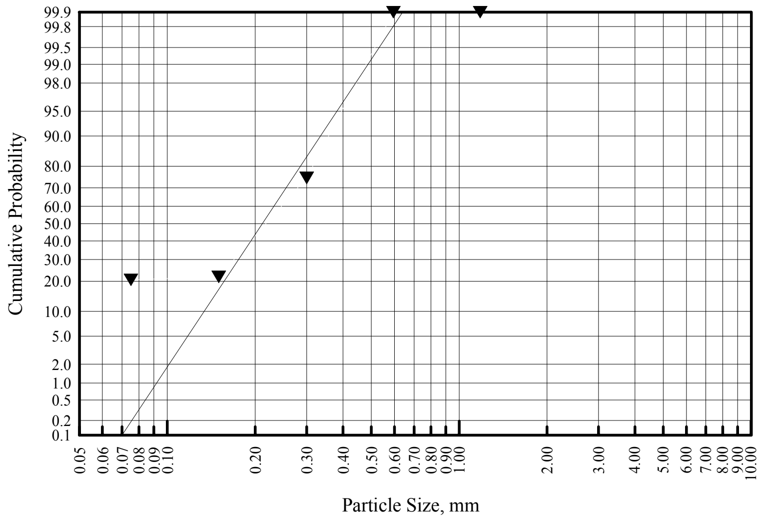
D_{10} = 0.16 mm,
D_{60} = 0.27 mm,
and
$C_u = D_{60}/ D_{10}$ = 1.69



At 6 ft. depth

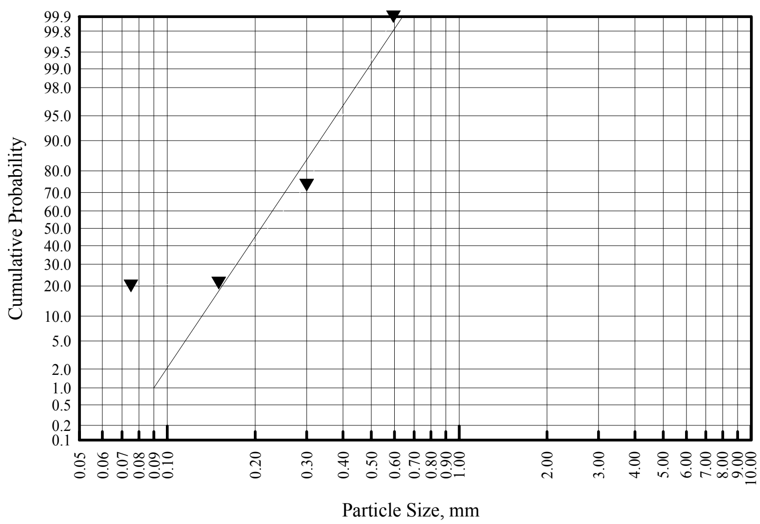
D_{10} = 0.18 mm,
D_{60} = 0.32 mm,
and
$C_u = D_{60}/ D_{10}$ = 1.78

Appendix I.2: Site 2, Monitoring Well Location #: 2 (Sample # 2/2)



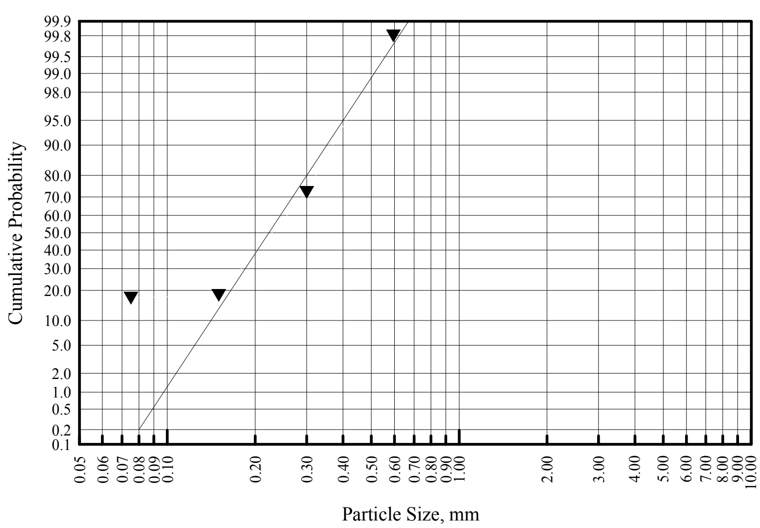
At 2 ft. depth

D_{10} = 0.15 mm,
D_{60} = 0.22 mm,
and
$C_u = D_{60}/ D_{10}$ = 1.47



At 4 ft. depth

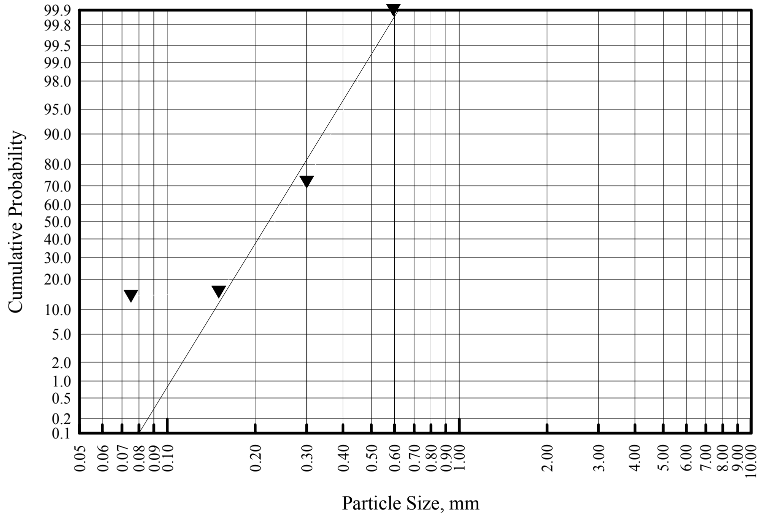
D_{10} = 0.23 mm,
D_{60} = 0.13 mm,
and
$C_u = D_{60}/ D_{10}$ = 1.77



At 6 ft. depth

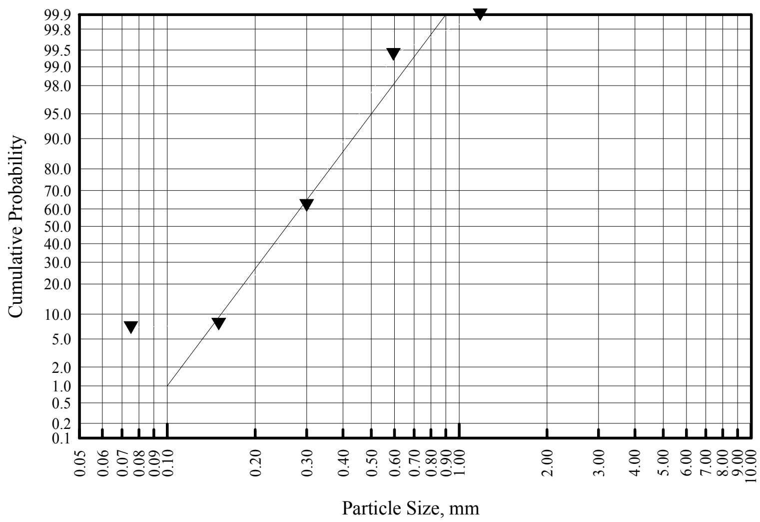
D_{10} = 0.14 mm,
D_{60} = 0.23 mm,
and
$C_u = D_{60}/ D_{10}$ = 1.64

Appendix I.2: Site 2, Monitoring Well Location #: 3 (Sample # 2/3)



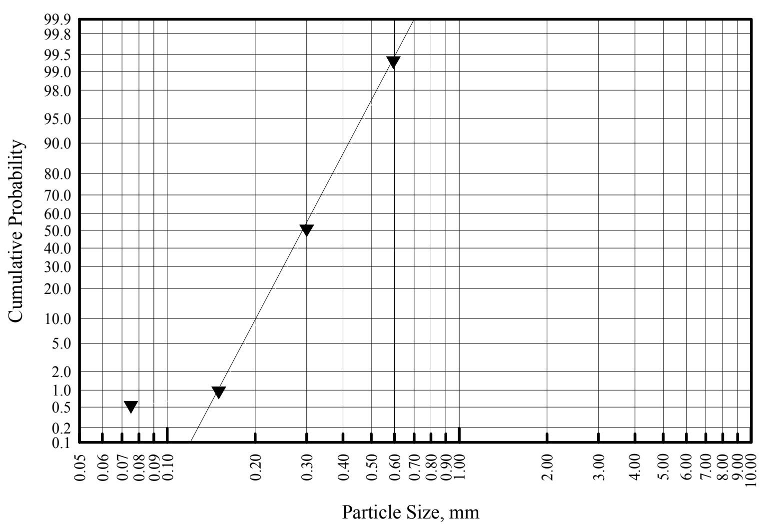
At 2 ft. depth

D_{10} = 0.14 mm,
D_{60} = 0.23 mm,
and
$C_u = D_{60}/ D_{10}$ = 1.64



At 4 ft. depth

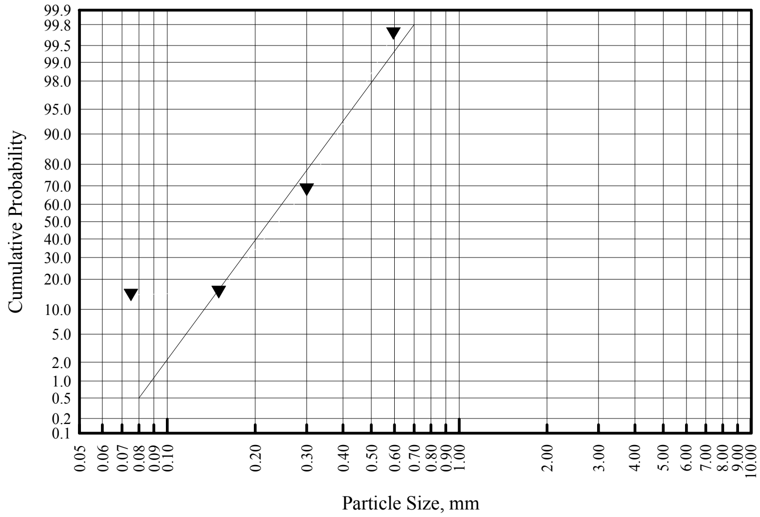
D_{10} = 0.15 mm,
D_{60} = 0.28 mm,
and
$C_u = D_{60}/ D_{10}$ = 1.87



At 6 ft. depth

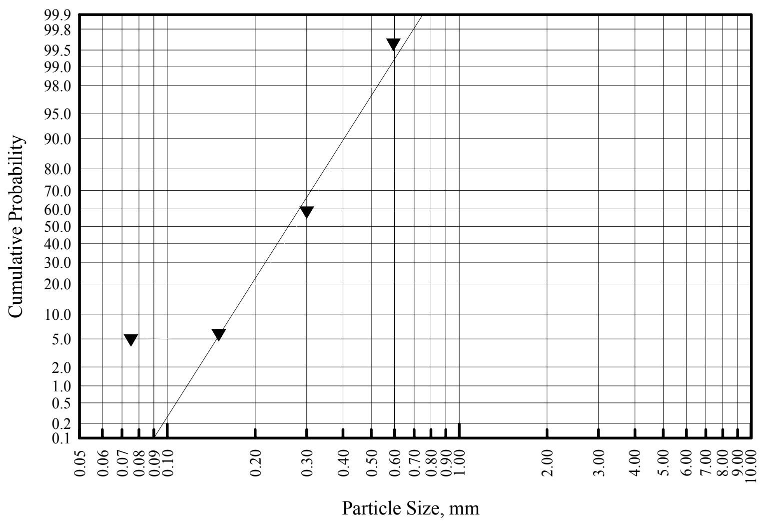
D_{10} = 0.20 mm,
D_{60} = 0.32 mm,
and
$C_u = D_{60}/ D_{10}$ = 1.60

Appendix I.2: Site 2, Monitoring Well Location #: 4 (Sample # 2/4)



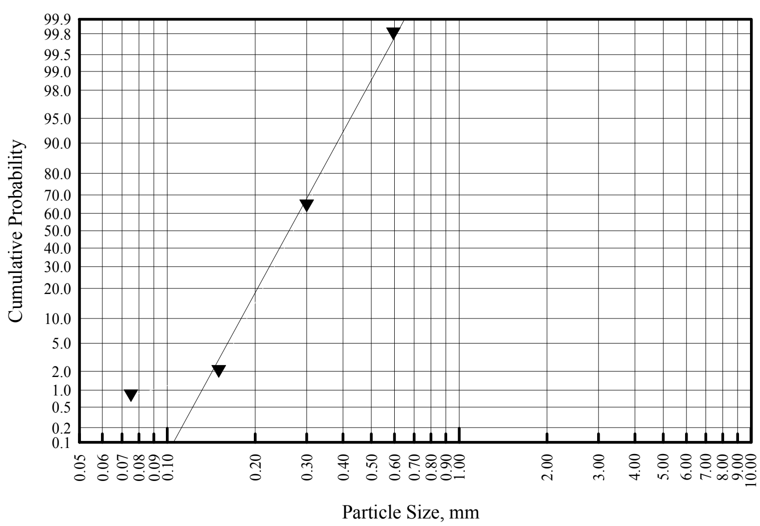
<u>At 2 ft. depth</u>

D_{10} = 0.13 mm,
D_{60} = 0.24 mm,
and
$C_u = D_{60}/ D_{10}$ = 1.85



<u>At 4 ft. depth</u>

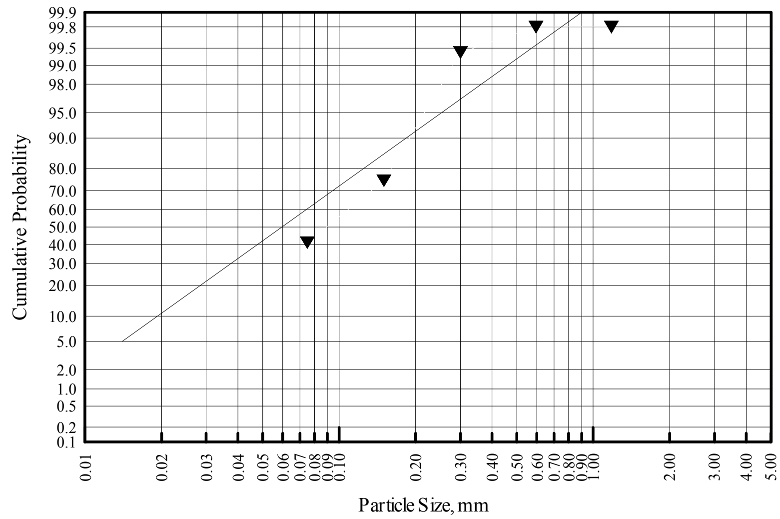
D_{10} = 0.14 mm,
D_{60} = 0.28 mm,
and
$C_u = D_{60}/ D_{10}$ = 2.00



<u>At 6 ft. depth</u>

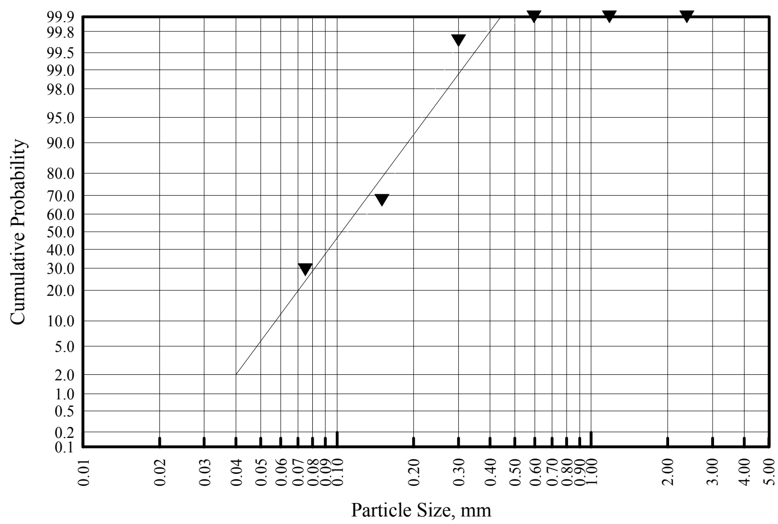
D_{10} = 0.17 mm,
D_{60} = 0.28 mm,
and
$C_u = D_{60}/ D_{10}$ = 1.65

Appendix I.3: Site 3, Monitoring Well Location #: 1 (Sample # 3/1)



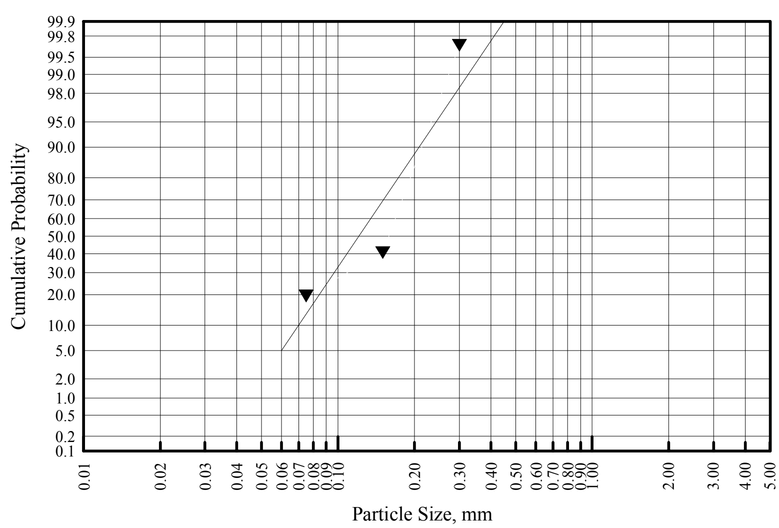
At 2 ft. depth

D_{10} = 0.045 mm,
D_{60} = 0.115 mm,
and
$C_u = D_{60}/ D_{10}$ = 2.56



At 4 ft. depth

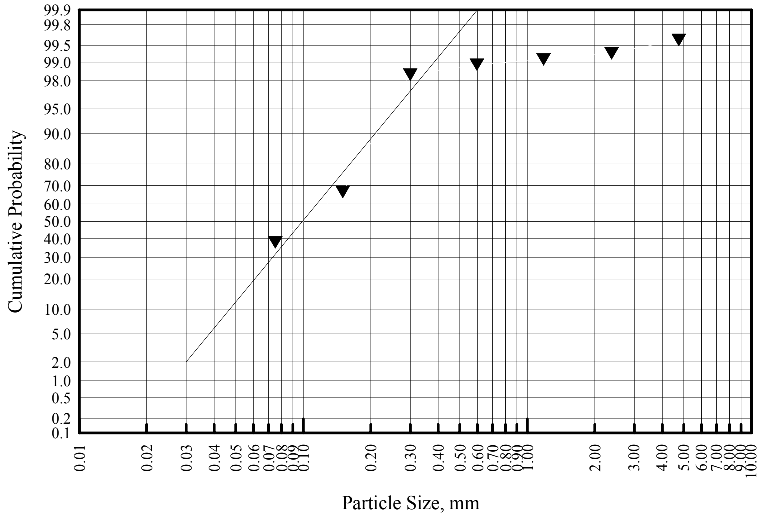
D_{10} = 0.058 mm,
D_{60} = 0.120 mm,
and
$C_u = D_{60}/ D_{10}$ = 2.07



At 6 ft. depth

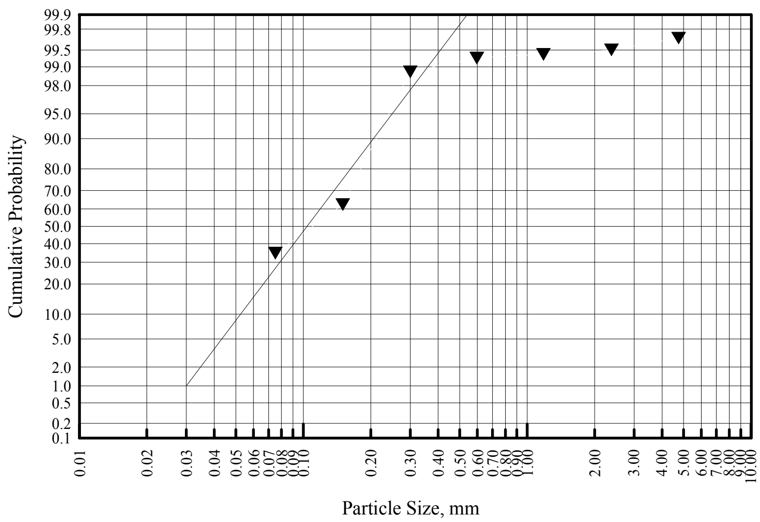
D_{10} = 0.07 mm,
D_{60} = 0.13 mm,
and
$C_u = D_{60}/ D_{10}$ = 1.86

Appendix I.3: Site 3, Monitoring Well Location #: 2 (Sample # 3/2)



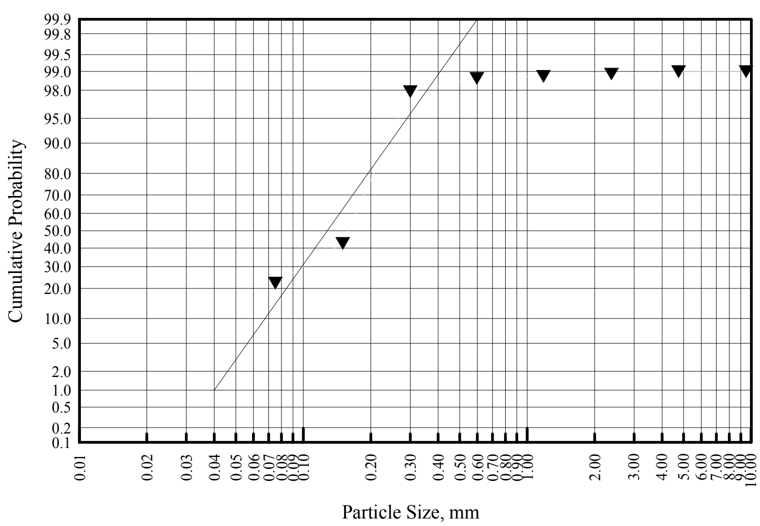
<u>At 2 ft. depth</u>

D_{10} = 0.048 mm,
D_{60} = 0.115 mm,
and
$C_u = D_{60}/ D_{10}$ = 2.45



<u>At 4 ft. depth</u>

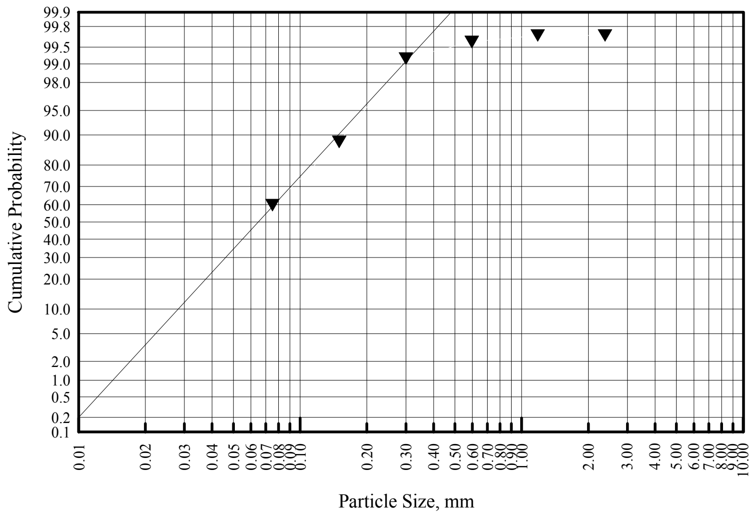
D_{10} = 0.053 mm,
D_{60} = 0.120 mm,
and
$C_u = D_{60}/ D_{10}$ = 2.26



<u>At 6 ft. depth</u>

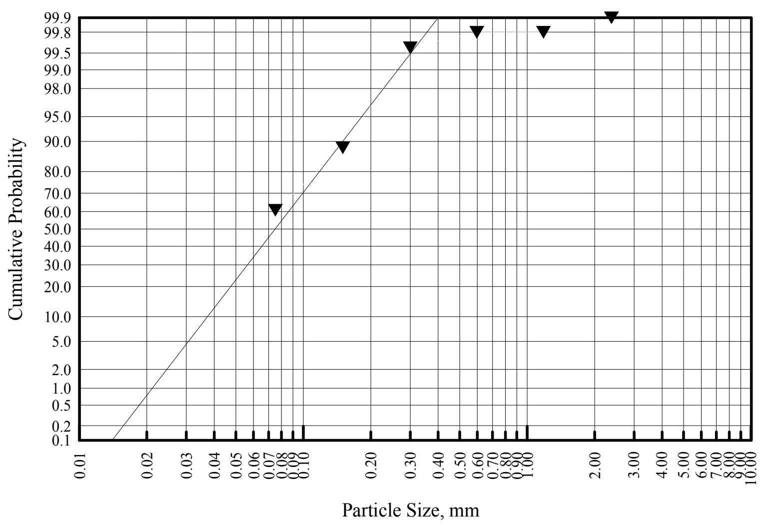
D_{10} = 0.066 mm,
D_{60} = 0.140 mm,
and
$C_u = D_{60}/ D_{10}$ = 2.12

Appendix I.3: Site 3, Monitoring Well Location #: 3 (Sample # 3/3)



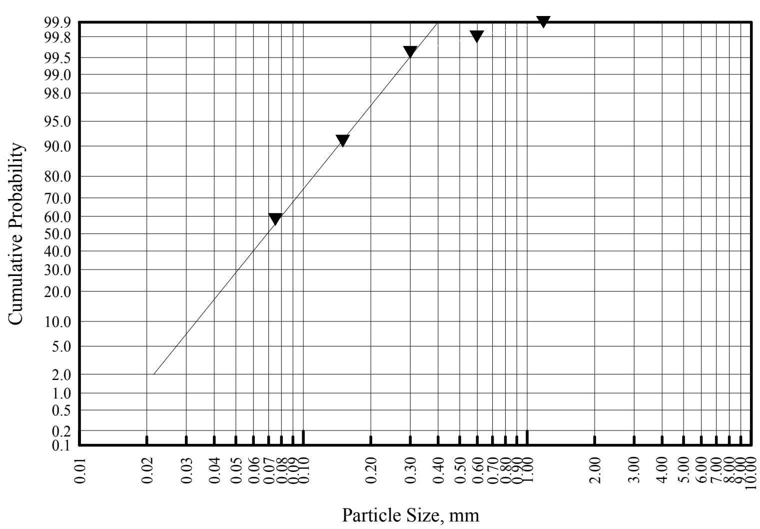
At 2 ft. depth

D_{10} = 0.028 mm,
D_{60} = 0.074 mm,
and
$C_u = D_{60}/ D_{10} = 2.64$



At 4 ft. depth

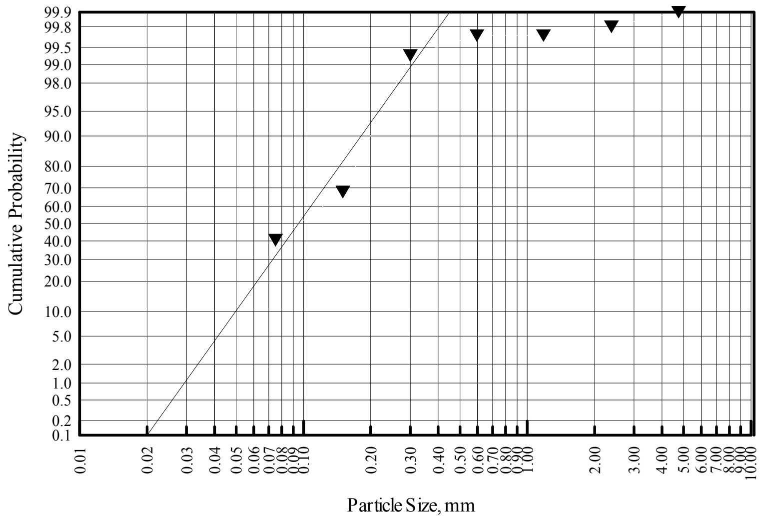
D_{10} = 0.037 mm,
D_{60} = 0.087 mm,
and
$C_u = D_{60}/ D_{10} = 2.35$



At 6 ft. depth

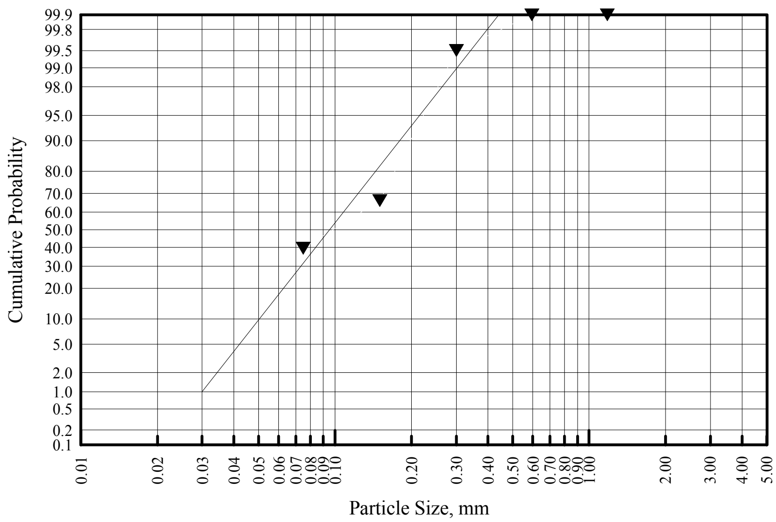
D_{10} = 0.032 mm,
D_{60} = 0.080 mm,
and
$C_u = D_{60}/ D_{10} = 2.50$

Appendix I.3: Site 3, Monitoring Well Location #: 4 (Sample # 3/4)



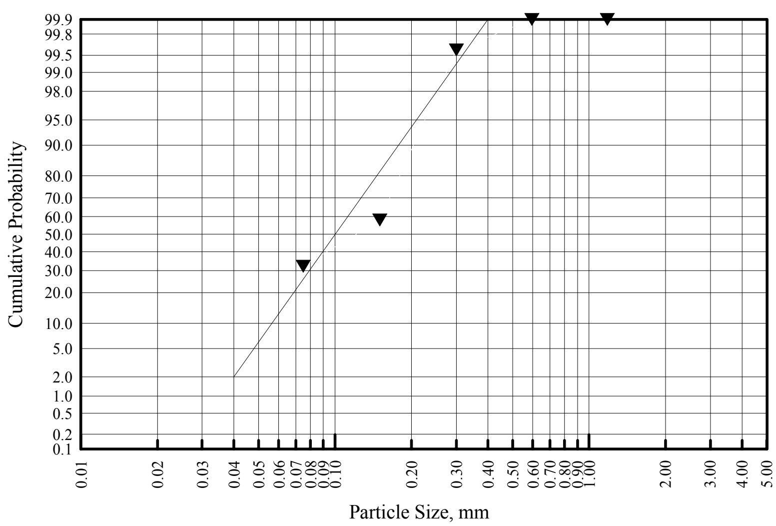
<u>At 2 ft. depth</u>

D_{10} = 0.050 mm,
D_{60} = 0.115 mm,
and
$C_u = D_{60}/ D_{10}$ = 2.30



<u>At 4 ft. depth</u>

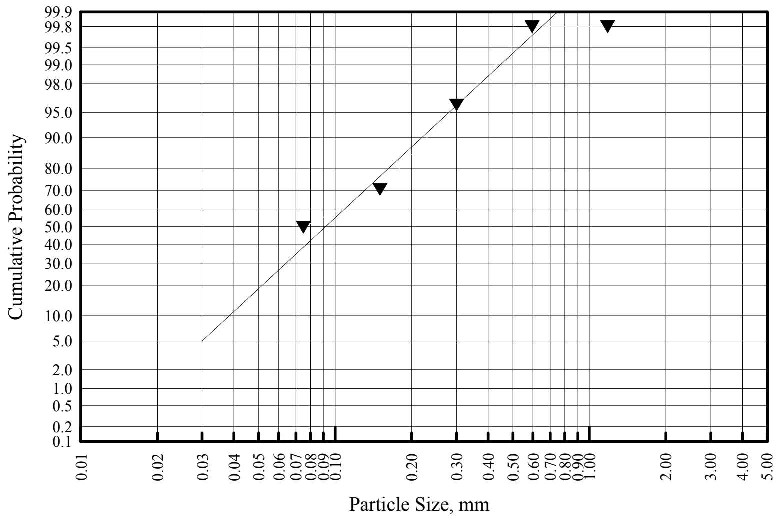
D_{10} = 0.050 mm,
D_{60} = 0.115 mm,
and
$C_u = D_{60}/ D_{10}$ = 2.30



<u>At 6 ft. depth</u>

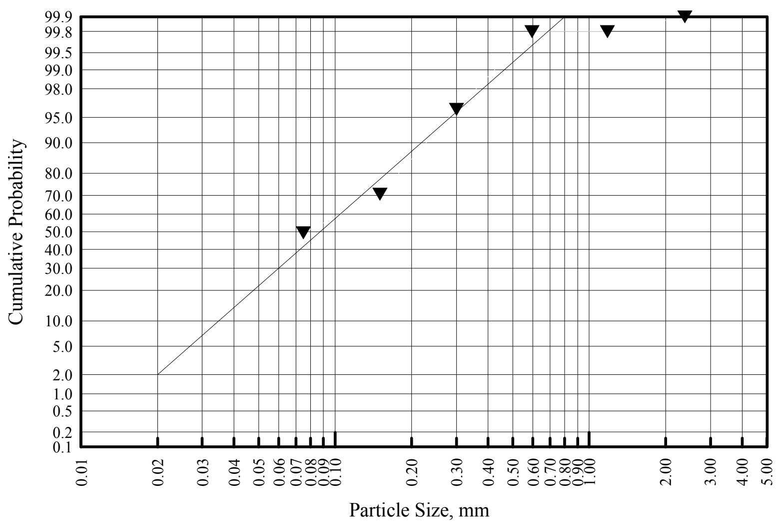
D_{10} = 0.57 mm,
D_{60} = 0.12 mm,
and
$C_u = D_{60}/ D_{10}$ = 2.10

Appendix I.4: Site 4, Monitoring Well Location #: 1 (Sample # 4/1)



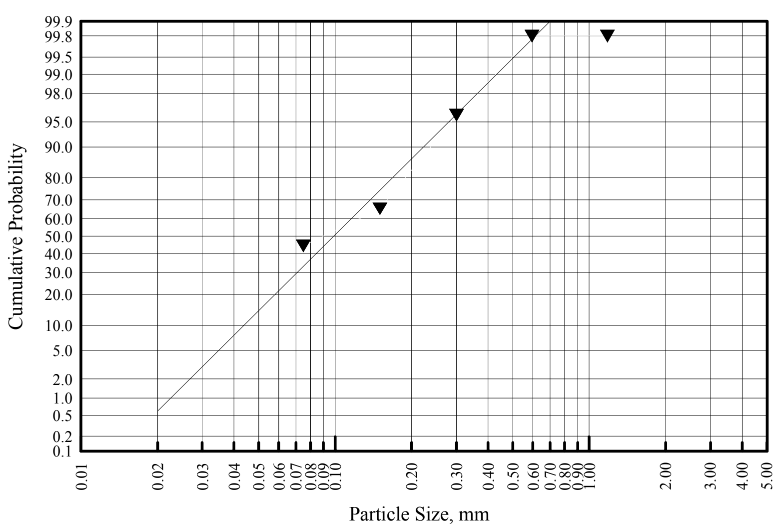
At 2 ft. depth

D_{10} = 0.038 mm,
D_{60} = 0.115 mm,
and
$C_u = D_{60}/ D_{10}$ = 3.03



At 4 ft. depth

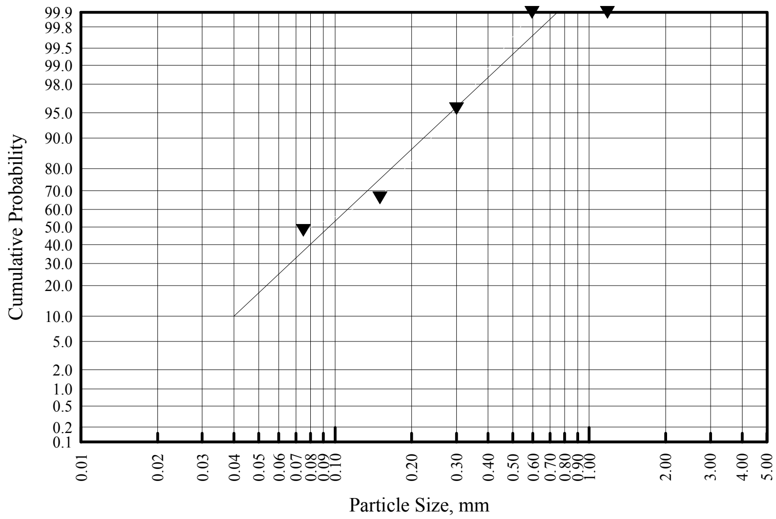
D_{10} = 0.034 mm,
D_{60} = 0.100 mm,
and
$C_u = D_{60}/ D_{10}$ = 2.94



At 6 ft. depth

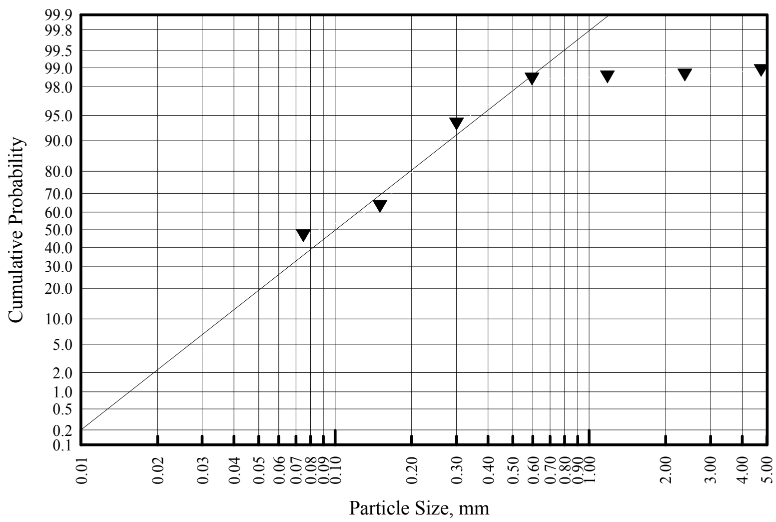
D_{10} = 0.042 mm,
D_{60} = 0.115 mm,
and
$C_u = D_{60}/ D_{10}$ = 2.74

Appendix I.4: Site 4, Monitoring Well Location #: 2 (Sample # 4/2)



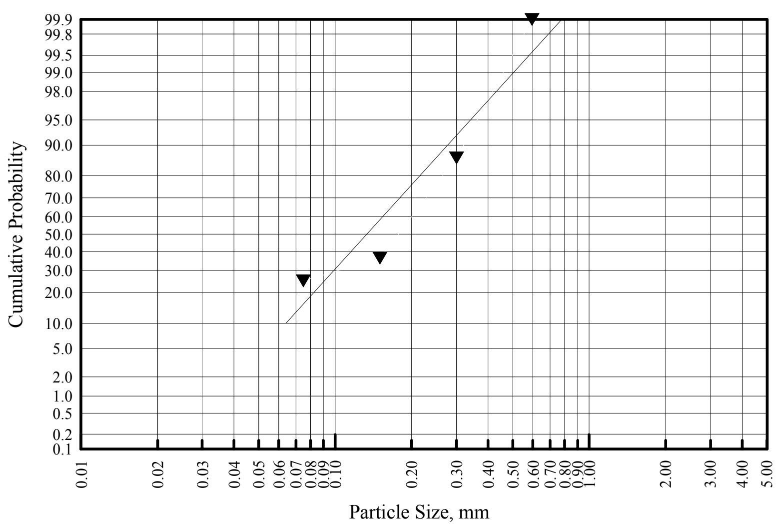
At 2 ft. depth

D_{10} = 0.040 mm,
D_{60} = 0.113 mm,
and
$C_u = D_{60}/ D_{10} = 2.83$



At 4 ft. depth

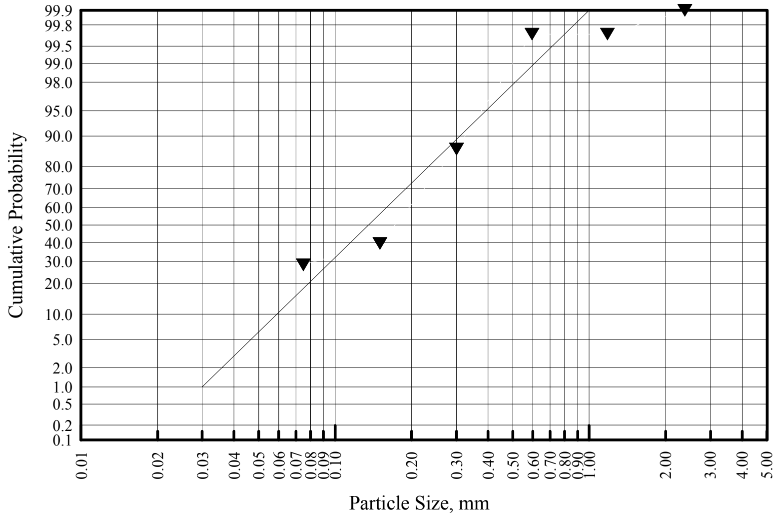
D_{10} = 0.035 mm,
D_{60} = 0.120 mm,
and
$C_u = D_{60}/ D_{10} = 3.43$



At 6 ft. depth

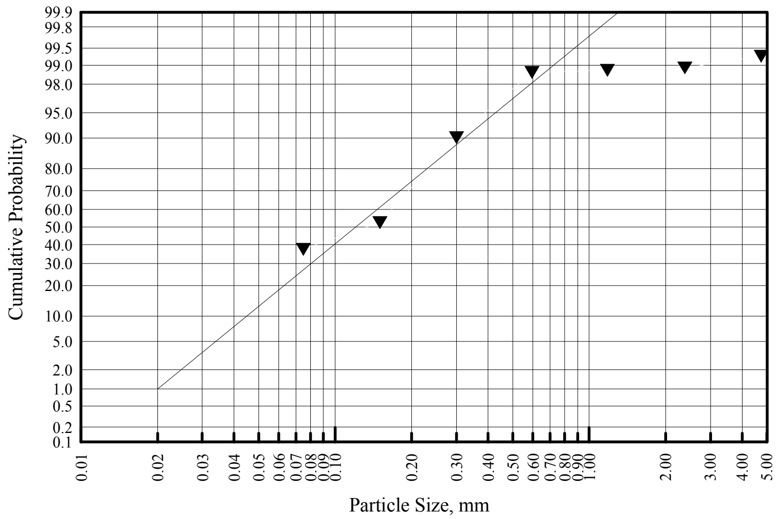
D_{10} = 0.063 mm,
D_{60} = 0.150 mm,
and
$C_u = D_{60}/ D_{10} = 2.38$

Appendix I.4: Site 4, Monitoring Well Location #: 3 (Sample # 4/3)



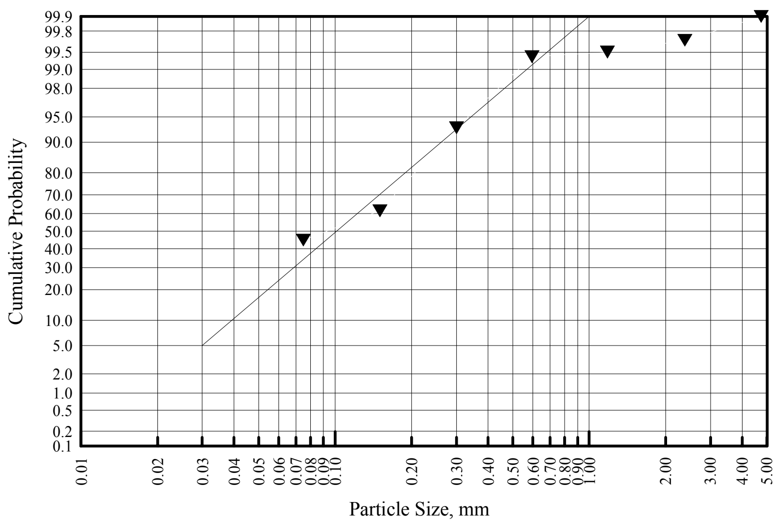
At 2 ft. depth

D_{10} = 0.068 mm,
D_{60} = 0.160 mm,
and
$C_u = D_{60}/ D_{10}$ = 2.35



At 4 ft. depth

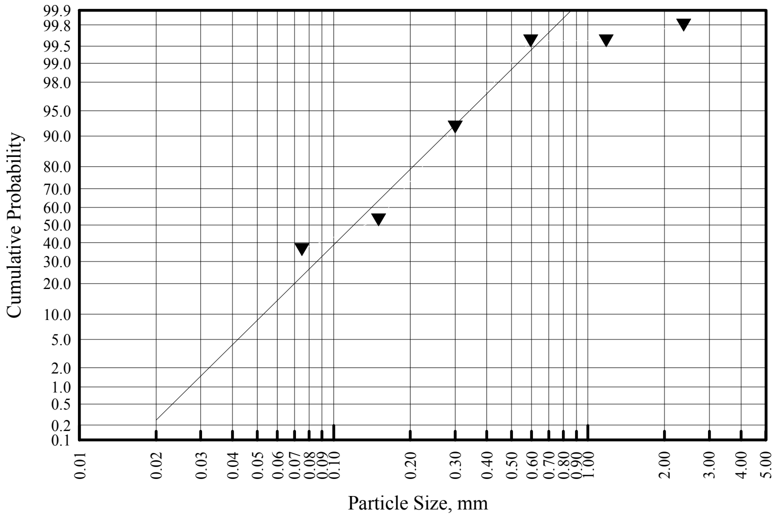
D_{10} = 0.043 mm,
D_{60} = 0.140 mm,
and
$C_u = D_{60}/ D_{10}$ = 3.26



At 6 ft. depth

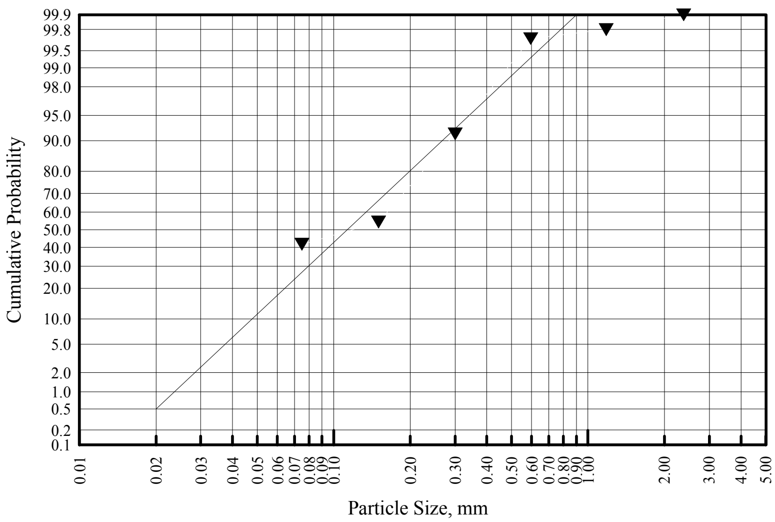
D_{10} = 0.039 mm,
D_{60} = 0.120 mm,
and
$C_u = D_{60}/ D_{10}$ = 3.08

Appendix I.4: Site 4, Monitoring Well Location #: 4 (Sample # 4/4)



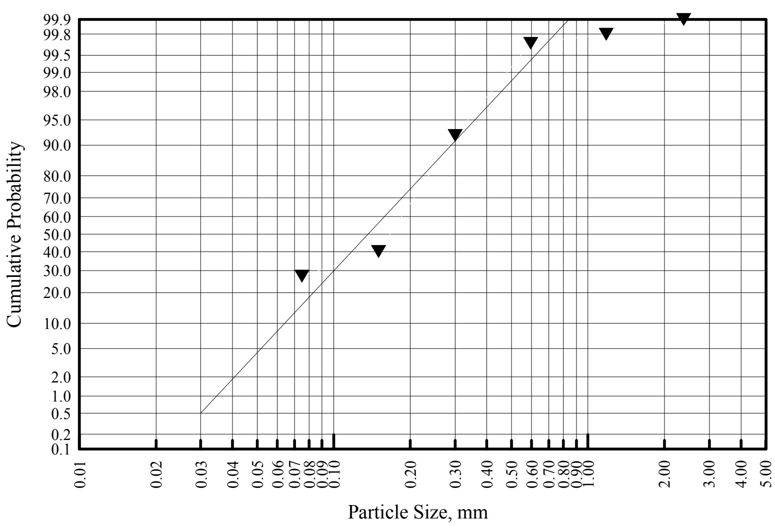
At 2 ft. depth

D_{10} = 0.052 mm,
D_{60} = 0.130 mm,
and
$C_u = D_{60}/ D_{10} = 2.5$



At 4 ft. depth

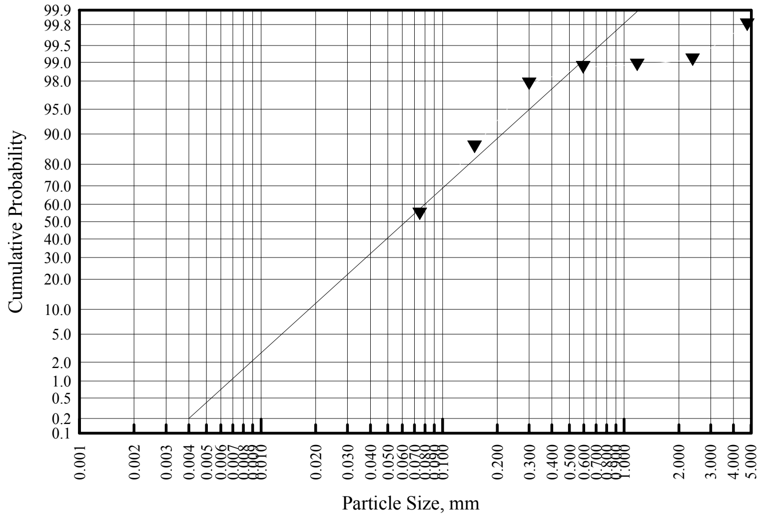
D_{10} = 0.048 mm,
D_{60} = 0.130 mm,
and
$C_u = D_{60}/ D_{10} = 2.71$



At 6 ft. depth

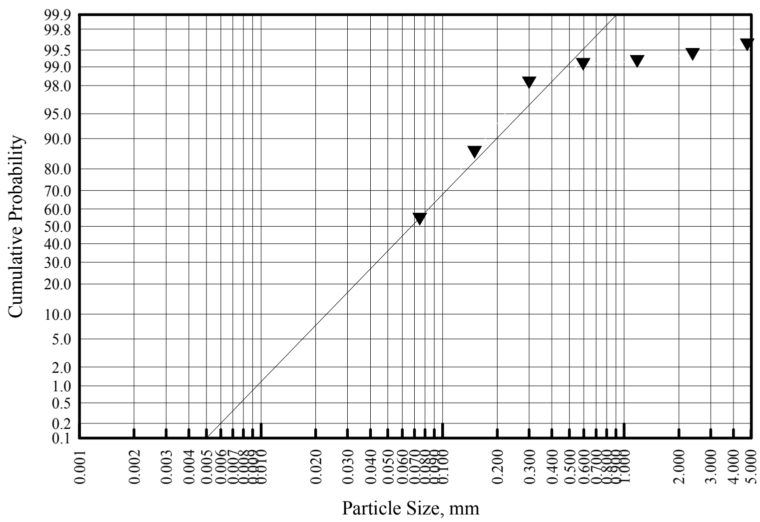
D_{10} = 0.062 mm,
D_{60} = 0.150 mm,
and
$C_u = D_{60}/ D_{10} = 2.42$

Appendix I.5: Site 5, Monitoring Well Location #: 1 (Sample # 5/1)



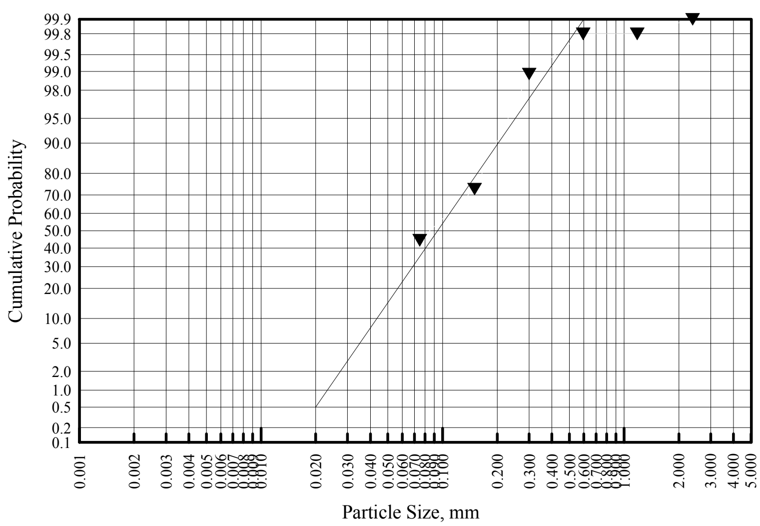
At 2 ft. depth

D_{10} = 0.017 mm,
D_{60} = 0.080 mm,
and
$C_u = D_{60}/ D_{10}$ = 4.71



At 4 ft. depth

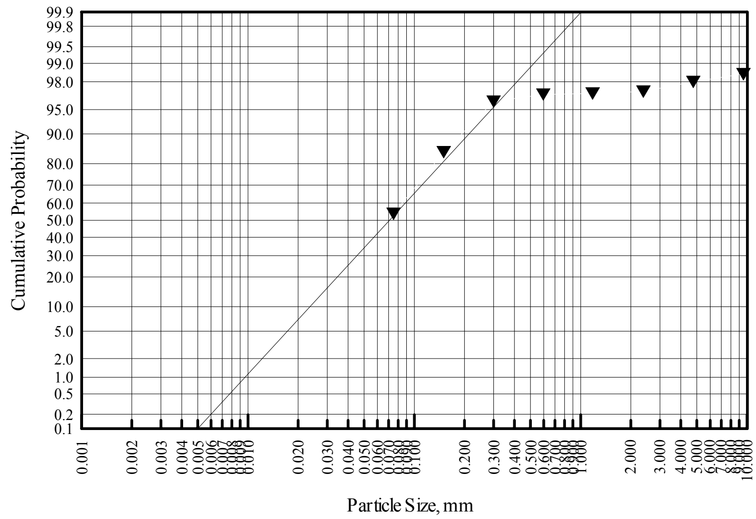
D_{10} = 0.022 mm,
D_{60} = 0.082 mm,
and
$C_u = D_{60}/ D_{10}$ = 3.73



At 6 ft. depth

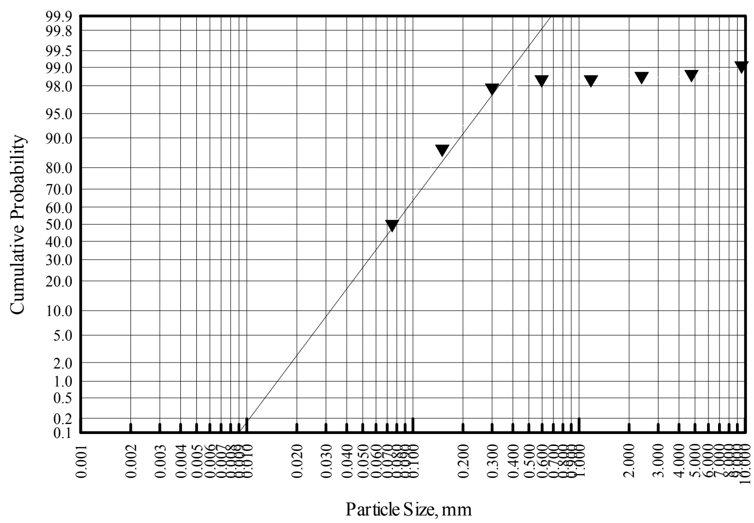
D_{10} = 0.042 mm,
D_{60} = 0.110 mm,
and
$C_u = D_{60}/ D_{10}$ = 2.62

Appendix I.5: Site 5, Monitoring Well Location #: 2 (Sample # 5/2)



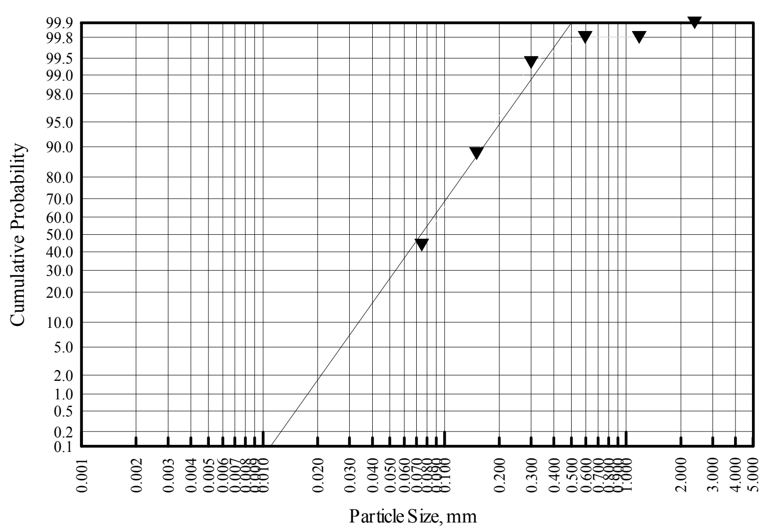
At 2 ft. depth

D_{10} = 0.022 mm,
D_{60} = 0.090 mm,
and
$C_u = D_{60}/ D_{10}$ = 4.09



At 4 ft. depth

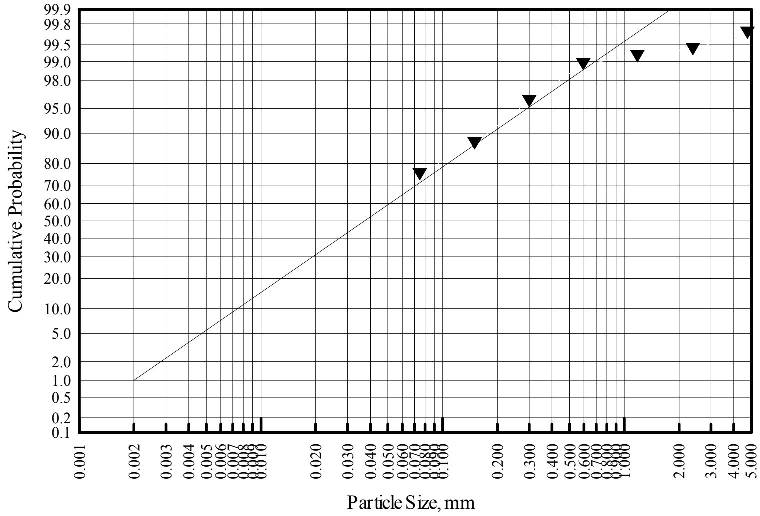
D_{10} = 0.031 mm,
D_{60} = 0.091 mm,
and
$C_u = D_{60}/ D_{10}$ = 2.94



At 6 ft. depth

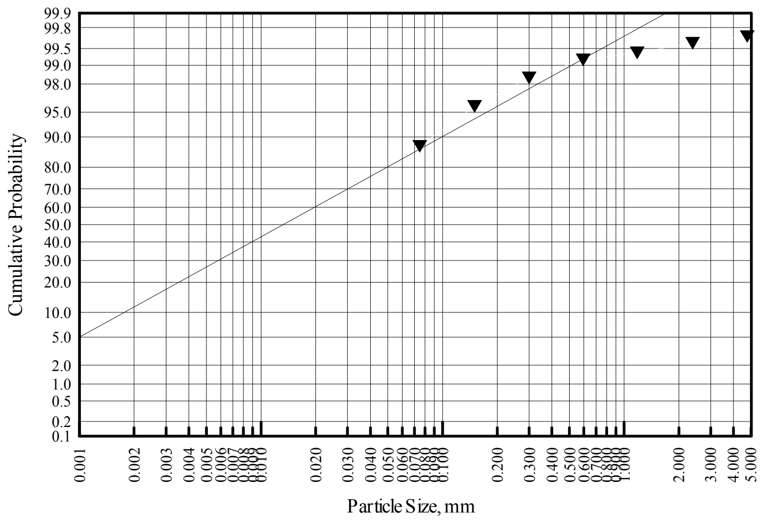
D_{10} = 0.032 mm,
D_{60} = 0.085 mm,
and
$C_u = D_{60}/ D_{10}$ = 2.66

Appendix I.5: Site 5, Monitoring Well Location #: 2 (Sample # 5/2)



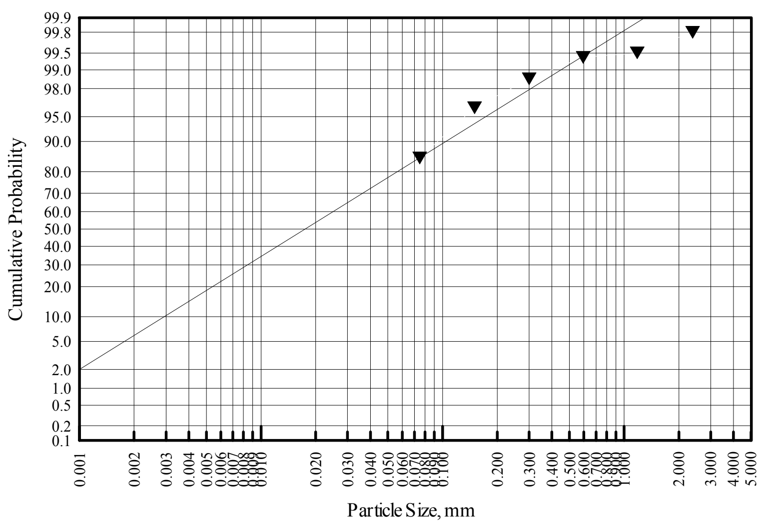
At 2 ft. depth

D_{10} = 0.007 mm,
D_{60} = 0.005 mm,
and
$C_u = D_{60}/ D_{10}$ = 7.14



At 4 ft. depth

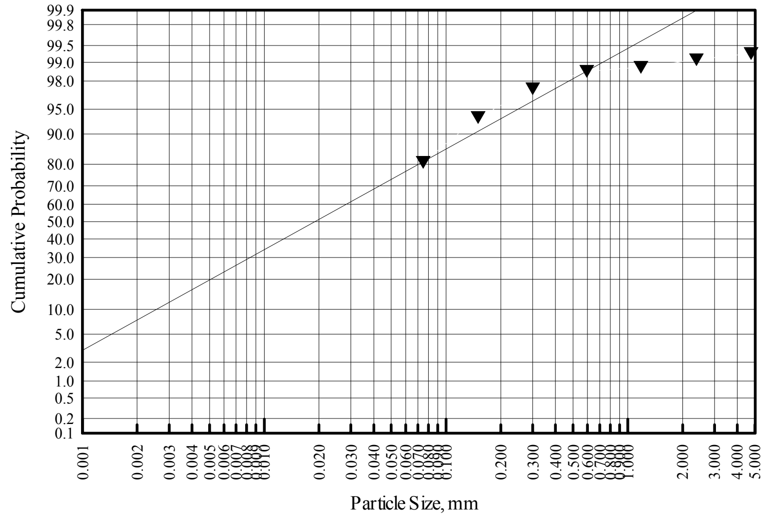
D_{10} = 0.0017 mm,
D_{60} = 0.019 mm,
and
$C_u = D_{60}/ D_{10}$ = 11.18



At 6 ft. depth

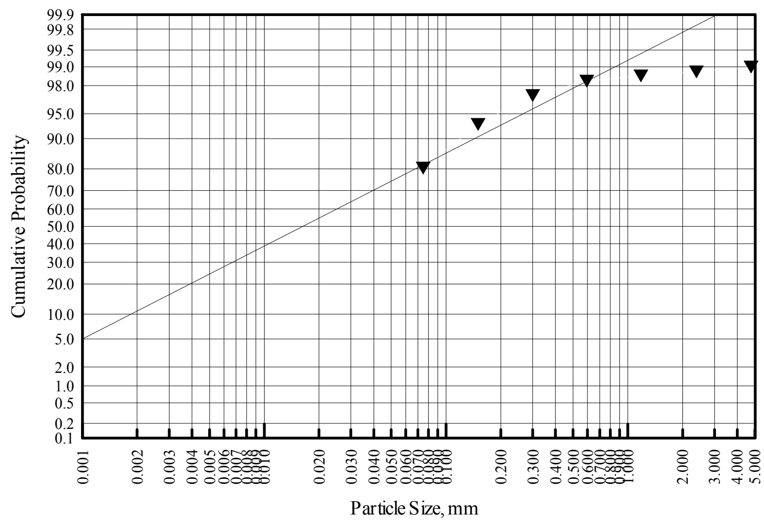
D_{10} = 0.003 mm,
D_{60} = 0.023 mm,
and
$C_u = D_{60}/ D_{10}$ = 7.67

Appendix I.5: Site 5, Monitoring Well Location #: 4 (Sample # 5/4)



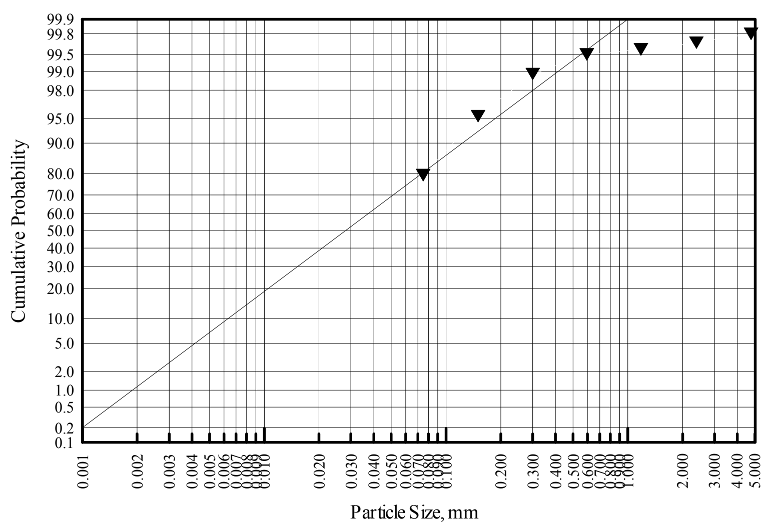
<u>At 2 ft. depth</u>

D_{10} = 0.0025 mm,
D_{60} = 0.028 mm,
and
$C_u = D_{60}/ D_{10}$ = 11.20



<u>At 4 ft. depth</u>

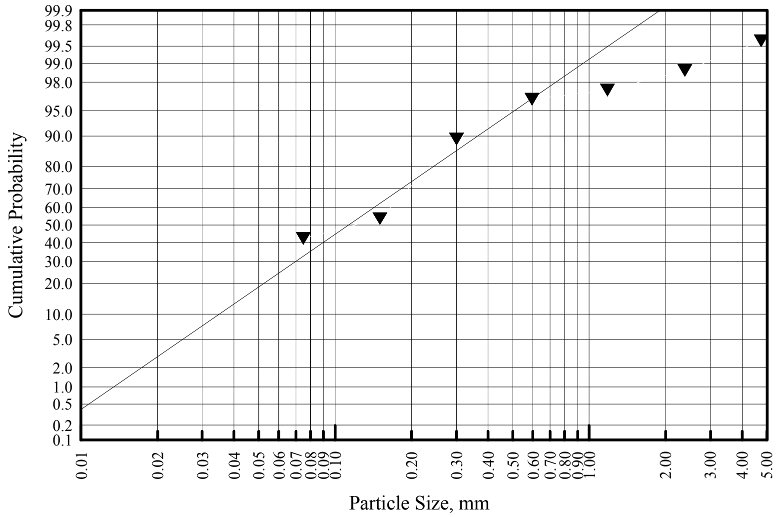
D_{10} = 0.0016 mm,
D_{60} = 0.020 mm,
and
$C_u = D_{60}/ D_{10}$ = 12.5



<u>At 6 ft. depth</u>

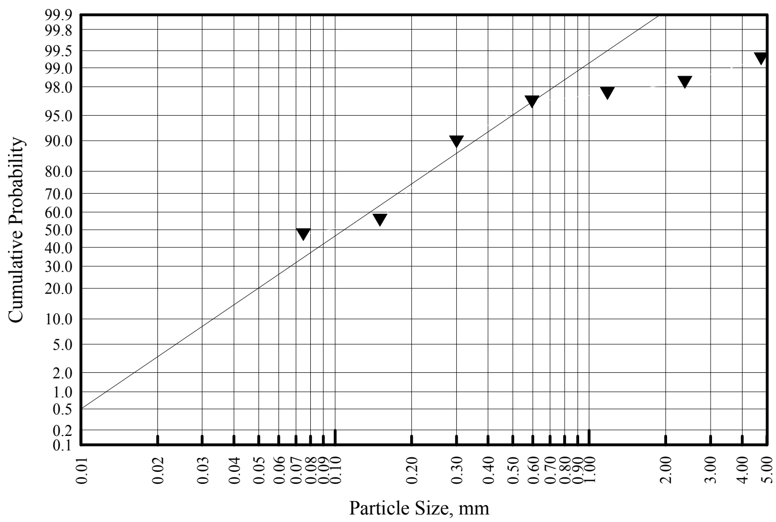
D_{10} = 0.0062 mm,
D_{60} = 0.036 mm,
and
$C_u = D_{60}/ D_{10}$ = 5.81

Appendix I.6: Site 6, Monitoring Well Location #: 1 (Sample # 6/1)



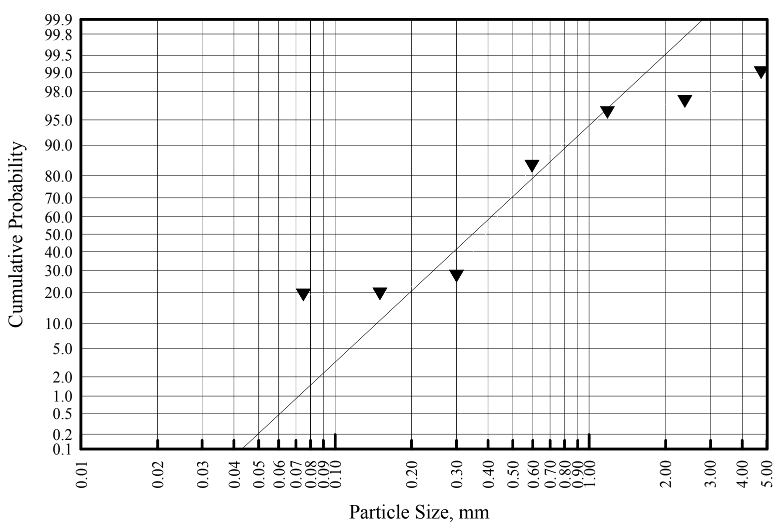
At 2 ft. depth

$D_{10} = 0.034$ mm,
$D_{60} = 0.140$ mm,
and
$C_u = D_{60}/ D_{10} = 4.12$



At 4 ft. depth

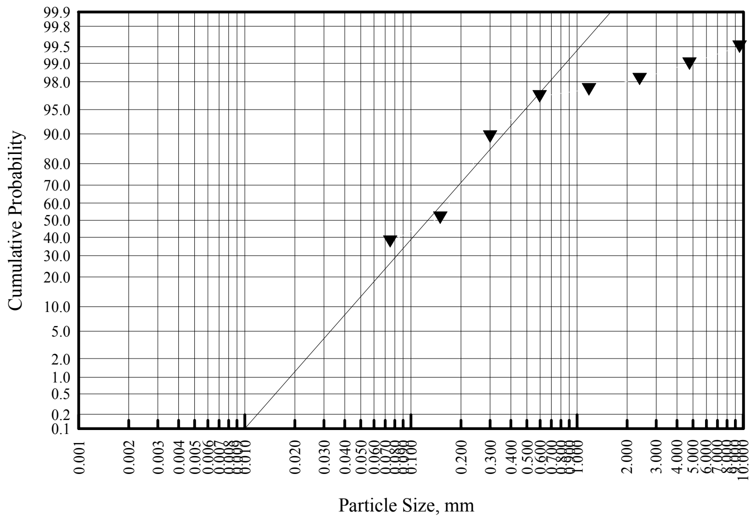
$D_{10} = 0.032$ mm,
$D_{60} = 0.130$ mm,
and
$C_u = D_{60}/ D_{10} = 4.06$



At 6 ft. depth

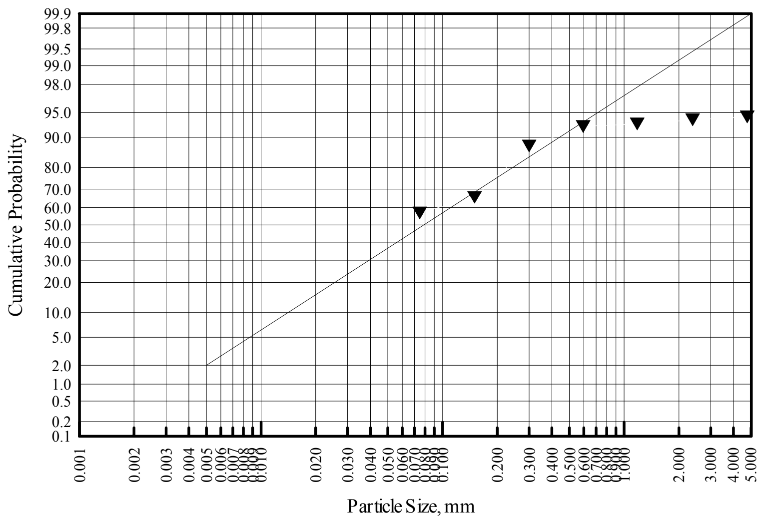
$D_{10} = 0.15$ mm,
$D_{60} = 0.41$ mm,
and
$C_u = D_{60}/ D_{10} = 2.73$

Appendix I.6: Site 6, Monitoring Well Location #: 2 (Sample # 6/2)



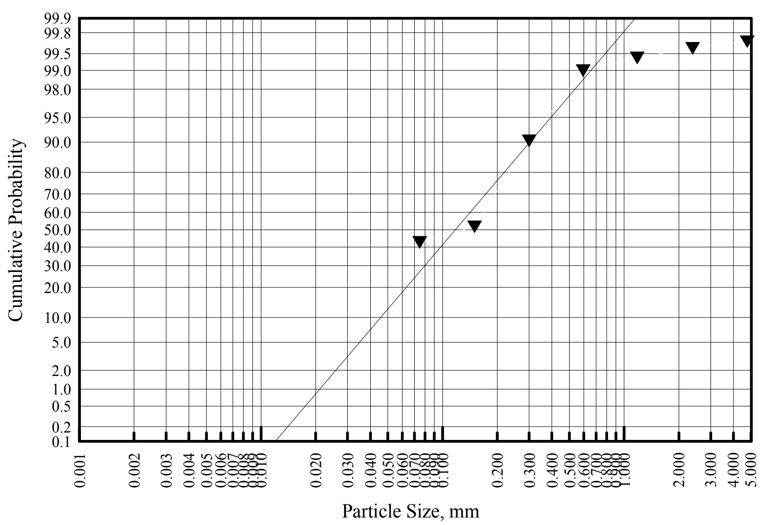
At 2 ft. depth

D_{10} = 0.044 mm,
D_{60} = 0.160 mm,
and
$C_u = D_{60}/ D_{10}$ = 3.64



At 4 ft. depth

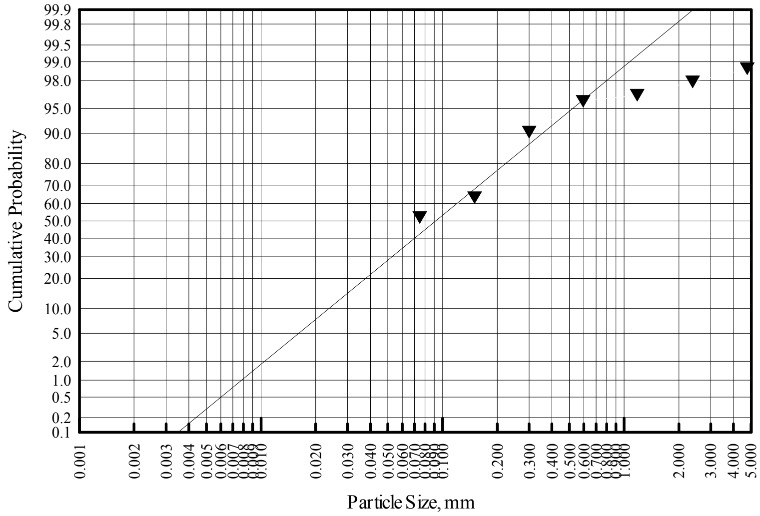
D_{10} = 0.014 mm,
D_{60} = 0.120 mm,
and
$C_u = D_{60}/ D_{10}$ = 8.57



At 6 ft. depth

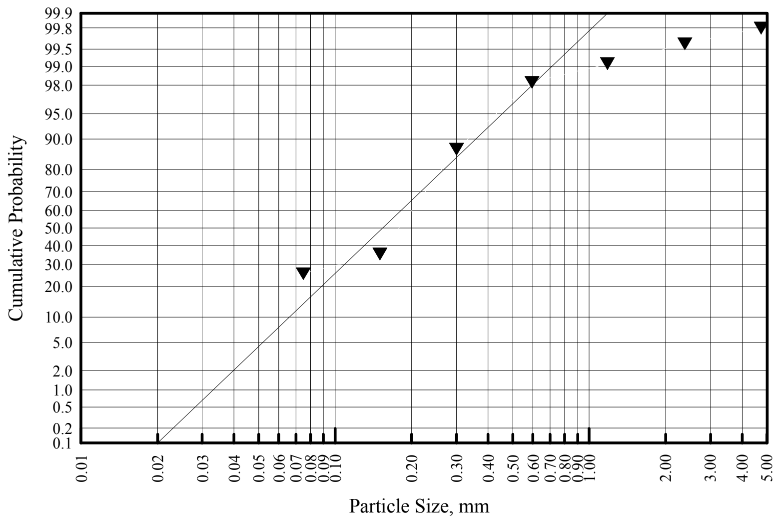
D_{10} = 0.044 mm,
D_{60} = 0.140 mm,
and
$C_u = D_{60}/ D_{10}$ = 3.18

Appendix I.6: Site 6, Monitoring Well Location #: 3 (Sample # 6/3)



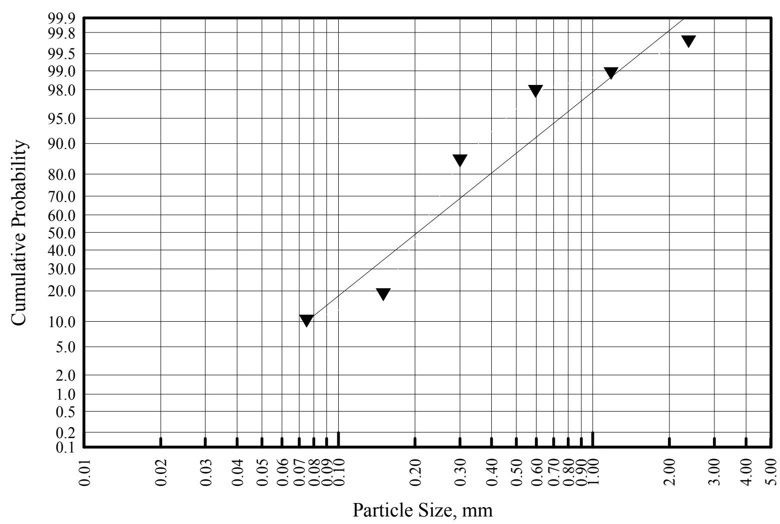
At 2 ft. depth

D_{10} = 0.023 mm,
D_{60} = 0.120 mm,
and
$C_u = D_{60}/ D_{10} = 5.22$



At 4 ft. depth

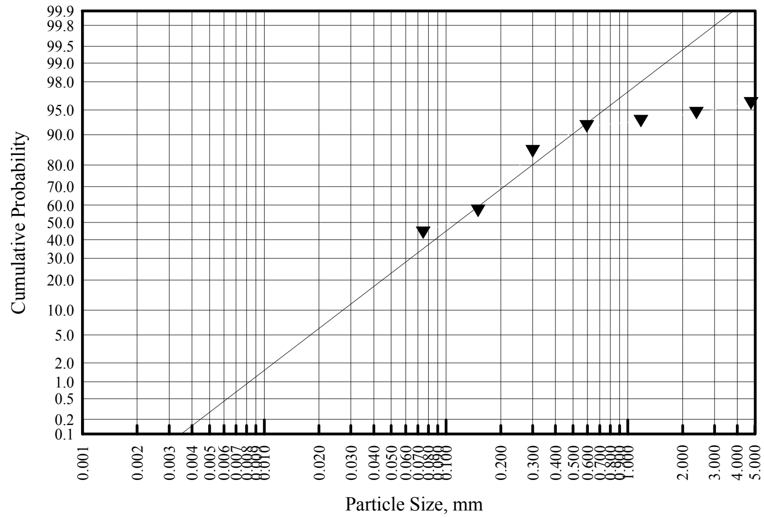
D_{10} = 0.065 mm,
D_{60} = 0.180 mm,
and
$C_u = D_{60}/ D_{10} = 2.77$



At 6 ft. depth

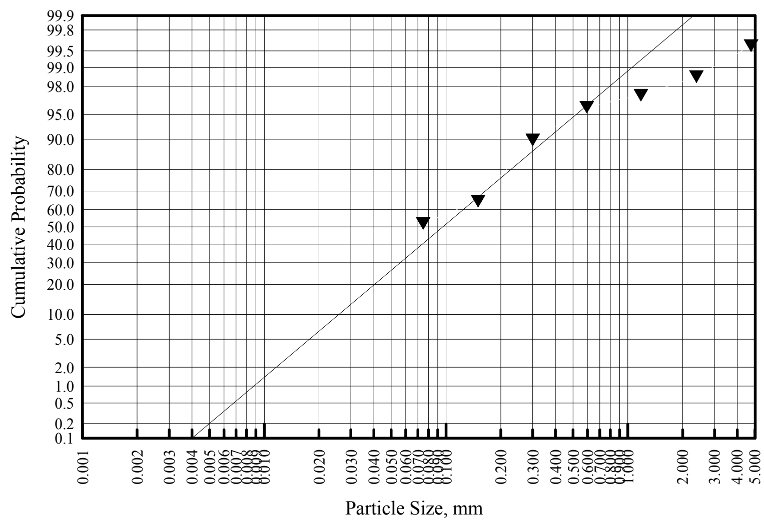
D_{10} = 0.010 mm,
D_{60} = 0.220 mm,
and
$C_u = D_{60}/ D_{10} = 2.20$

Appendix I.6: Site 6, Monitoring Well Location #: 4 (Sample # 6/4)



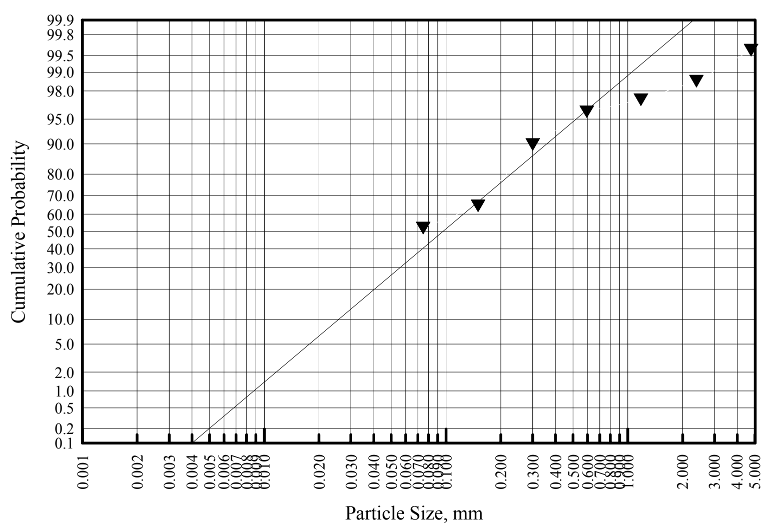
<u>At 2 ft. depth</u>

D_{10} = 0.026 mm,
D_{60} = 0.140 mm,
and
$C_u = D_{60}/ D_{10}$ = 5.39



<u>At 4 ft. depth</u>

D_{10} = 0.024 mm,
D_{60} = 0.120 mm,
and
$C_u = D_{60}/ D_{10}$ = 5.00



<u>At 6 ft. depth</u>

D_{10} = 0.032 mm,
D_{60} = 0.120 mm,
and
$C_u = D_{60}/ D_{10}$ = 3.75

APPENDIX J

SOIL CHARACTERIZATION DATA TABLE

Appendix J.1: Soil Classification Data for Site 1 (Drip Irrigation)

Location[a] #	Depth[b] ft	LL, %	PL %	PI, %	d_{10}, mm	d_{60}, mm	C_u	ρ	K, cm/s	Soil Classification
7	2		NP		0.14	0.25	1.79	0.54	0.1184	Loamy sand
	4		NP		0.15	0.26	1.73	0.43	0.0447	Medium to fine sand
	6		NP		0.14	0.26	1.86	0.41	0.0315	Medium to fine sand
8	2		NP		0.14	0.28	2.00	0.45	0.0479	Medium to fine sand
	4		NP		0.15	0.30	2.00	0.43	0.0447	Medium to fine sand
	6		NP		0.20	0.33	1.65	0.41	0.0643	Medium to fine sand
9	2		NP		0.14	0.27	1.92	0.44	0.0432	Medium to fine sand
	4		NP		0.13	0.26	2.00	0.43	0.0336	Medium to fine sand
	6		NP		0.16	0.30	1.88	0.44	0.0565	Medium to fine sand
10	2		NP		0.13	0.26	2.00	0.44	0.0373	Medium to fine sand
	4		NP		0.14	0.25	1.79	0.44	0.0432	Medium to fine sand
	6		NP		0.15	0.26	1.73	0.40	0.0325	Medium to fine sand

[a] Refer to the layout plan, [b] Depth of soil sampling

LL – Liquid Limit, PL – Plastic Limit, PI – Plasticity Index, d_{10} - Grain size that is 10% finer by weight (effective size), d_{60} - Grain size that is 60% finer by weight, C_u - Uniformity Coefficient, ρ – Porosity, K - Hydraulic Conductivity, NP – Non Plastic

Appendix J.2: Soil Classification Data for Site 2 (Drip Irrigation)

Location[a] #	Depth[b] ft	LL, %	PL %	PI, %	d_{10}, mm	d_{60}, mm	C_u	ρ	K, cm/s	Soil Classification
7	2		NP		0.12	0.26	2.17	0.45	0.0352	Medium to fine sand
	4		NP		0.16	0.27	1.69	0.43	0.0508	Medium to fine sand
	6		NP		0.18	0.32	1.78	0.42	0.0579	Medium to fine sand
8	2		NP		0.15	0.22	1.47	0.45	0.0550	Loamy sand
	4		NP		0.13	0.23	1.77	0.44	0.1167	Loamy sand
	6		NP		0.14	0.23	1.64	0.45	0.0479	Loamy sand
9	2		NP		0.14	0.23	1.64	0.51	0.0879	Medium to fine sand
	4		NP		0.15	0.28	1.87	0.55	0.1501	Medium to fine sand
	6		NP		0.20	0.32	1.60	0.42	0.0715	Medium to fine sand
10	2		NP		0.13	0.24	1.85	0.47	0.0507	Medium to fine sand
	4		NP		0.14	0.28	2.00	0.42	0.0350	Medium to fine sand
	6		NP		0.17	0.28	1.65	0.40	0.0417	Medium to fine sand

[a] Refer to the layout plan, [b] Depth of soil sampling

LL – Liquid Limit, PL – Plastic Limit, PI – Plasticity Index, d_{10} - Grain size that is 10% finer by weight (effective size), d_{60} - Grain size that is 60% finer by weight, C_u - Uniformity Coefficient, ρ – Porosity, K - Hydraulic Conductivity, NP – Non Plastic

Appendix J.3: Soil Classification Data for Site 3 (Sprinkler Irrigation)

Location[a] #	Depth[b] ft	LL, %	PL %	PI, %	d_{10}, mm	d_{60}, mm	C_u	ρ	K, cm/s	Soil Classification
7	2		NP		0.045	0.115	2.56	0.46	0.0055	Sandy loam
	4		NP		0.058	0.120	2.07	0.45	0.0082	Loamy sand
	6		NP		0.070	0.130	1.86	0.51	0.0220	Loamy sand
8	2		NP		0.048	0.115	2.45	0.51	0.0103	Sandy loam
	4		NP		0.053	0.120	2.26	0.51	0.0126	Sandy loam
	6		NP		0.066	0.140	2.12	0.51	0.0195	Loamy sand
9	2		NP		0.028	0.074	2.64	0.51	0.0035	Silt loam
	4		NP		0.037	0.087	2.35	0.51	0.0061	Silt loam
	6		NP		0.032	0.080	2.50	0.52	0.0051	Silt loam
10	2		NP		0.050	0.115	2.30	0.51	0.0112	Sandy loam
	4		NP		0.050	0.115	2.30	0.48	0.0083	Sandy loam
	6		NP		0.570	0.120	2.10	0.51	1.4572	Sandy loam

[a] Refer to the layout plan, [b] Depth of soil sampling

LL – Liquid Limit, PL – Plastic Limit, PI – Plasticity Index, d_{10} - Grain size that is 10% finer by weight (effective size), d_{60} - Grain size that is 60% finer by weight, C_u - Uniformity Coefficient, ρ – Porosity, K - Hydraulic Conductivity, NP – Non Plastic

Appendix J.4: Soil Classification Data for Site 4 (Sprinkler Irrigation)

Location[a] #	Depth[b] ft	LL, %	PL %	PI, %	d_{10}, mm	d_{60}, mm	C_u	ρ	K, cm/s	Soil Classification
7	2		NP		0.038	0.115	3.03	0.50	0.0059	Sandy loam
	4		NP		0.034	0.100	2.94	0.53	0.0063	Sandy loam
	6		NP		0.042	0.115	2.74	0.50	0.0072	Sandy loam
8	2		NP		0.040	0.113	2.83	0.51	0.0072	Sandy loam
	4		NP		0.035	0.120	3.43	0.51	0.0055	Sandy loam
	6		NP		0.063	0.150	2.38	0.54	0.0240	Loamy sand
9	2		NP		0.068	0.160	2.35	0.51	0.0207	Loamy sand
	4		NP		0.043	0.140	3.26	0.51	0.0083	Sandy loam
	6		NP		0.039	0.120	3.08	0.47	0.0046	Sandy loam
10	2		NP		0.052	0.130	2.50	0.51	0.0121	Sandy loam
	4		NP		0.048	0.130	2.71	0.49	0.0085	Sandy loam
	6		NP		0.062	0.150	2.42	0.51	0.0172	Loamy sand

[a] Refer to the layout plan, [b] Depth of soil sampling

LL – Liquid Limit, PL – Plastic Limit, PI – Plasticity Index, d_{10} - Grain size that is 10% finer by weight (effective size), d_{60} - Grain size that is 60% finer by weight, C_u - Uniformity Coefficient, ρ – Porosity, K - Hydraulic Conductivity, NP – Non Plastic

Appendix J.5: Soil Classification Data for Site 5 (Sand Mound)

Location[a] #	Depth[b] ft	LL, %	PL %	PI, %	d_{10}, mm	d_{60}, mm	C_u	ρ	K, cm/s	Soil Classification
7	2	28	18	10	0.0170	0.080	4.71	0.56	0.00117	Low plasticity clay
	4	35	19	16	0.0220	0.082	3.73	0.60	0.00265	Low plasticity clay
	6	36	18	18	0.0420	0.110	2.62	0.58	0.00965	Low plasticity clay
8	2	29	20	9	0.0220	0.090	4.09	0.59	0.00265	Low plasticity clay
	4	30	20	10	0.0310	0.091	2.94	0.60	0.00526	Low plasticity clay
	6	32	20	12	0.0320	0.085	2.66	0.64	0.00560	Low plasticity clay
9	2	41	20	21	0.0070	0.050	7.14	0.56	0.00027	Low plasticity clay
	4	56	23	33	0.0017	0.019	11.18	0.58	0.00002	High plasticity clay
	6	56	22	34	0.0030	0.023	7.67	0.61	0.00005	High plasticity clay
10	2	32	23	9	0.0025	0.028	11.20	0.59	0.00003	Low plasticity clay
	4	39	20	19	0.0016	0.020	12.50	0.59	0.00001	Low plasticity clay
	6	46	16	30	0.0062	0.036	5.81	0.54	0.00021	Low plasticity clay

[a] Refer to the layout plan, [b] Depth of soil sampling

LL – Liquid Limit, PL – Plastic Limit, PI – Plasticity Index, d_{10} - Grain size that is 10% finer by weight (effective size), d_{60} - Grain size that is 60% finer by weight, C_u - Uniformity Coefficient, ρ – Porosity, K - Hydraulic Conductivity, NP – Non Plastic

Appendix J.6: Soil Classification Data for Site 6 (Sand Mound)

Location[a] #	Depth[b] ft	LL, %	PL %	PI, %	d_{10}, mm	d_{60}, mm	C_u	ρ	K, cm/s	Soil Classification
7	2	18	13	5	0.034	0.14	4.12	0.51	0.0047	Low plasticity silty clay
	4	36	15	21	0.032	0.13	4.06	0.55	0.0042	Low plasticity clay
	6		NP		0.150	0.41	2.73	0.48	0.0913	Loamy sand
8	2		NP		0.044	0.16	3.64	0.46	0.0079	Sandy loam
	4	35	19	17	0.014	0.12	8.57	0.56	0.0008	Low plasticity clay
	6	31	14	17	0.044	0.14	3.18	0.55	0.0079	Low plasticity clay
9	2	18	15	2	0.023	0.12	5.22	0.48	0.0022	Low plasticity organic silt and clay
	4		NP		0.065	0.17	2.77	0.50	0.0172	Loamy sand
	6		NP		0.100	0.22	2.20	0.43	0.0406	Sand
10	2	22	21	1	0.026	0.14	5.39	0.54	0.0027	Low plasticity organic silt and clay
	4	26	15	11	0.024	0.12	5.00	0.56	0.0023	Low plasticity clay
	6	36	13	23	0.032	0.12	3.75	0.56	0.0042	Low plasticity clay

[a] Refer to the layout plan, [b] Depth of soil sampling

LL – Liquid Limit, PL – Plastic Limit, PI – Plasticity Index, d_{10} - Grain size that is 10% finer by weight (effective size), d_{60} - Grain size that is 60% finer by weight, C_u - Uniformity Coefficient, ρ – Porosity, K - Hydraulic Conductivity, NP – Non Plastic

APPENDIX K

WATER ELEVATION LEVEL DATA IN MONITORING WELLS

Appendix K.1: Recorded Water Levels in Monitoring Wells in Drip Irrigation Sites

(Sites 1 and 2)

	Site 1				Site 2			
	Monitoring Well Location				Monitoring Well Location			
	7	8	9	10	7	8	9	10
Trip No.	Water Surface Elevations							
1	a	a	a	a	a	a	a	a
3	a	a	a	a	13.92	14.01	13.98	13.96
5	a	a	a	a	14.77	14.91	14.83	14.86
7	a	a	a	a	14.82	14.86	14.93	14.96
9	a	a	a	a	14.72	14.71	14.78	14.81
11	a	a	a	a	14.22	14.26	14.33	14.36
13	a	a	a	a	15.27	15.31	15.38	15.41
15	13.13	13.08	13.08	a	16.42	16.46	16.43	16.56
17	14.73	14.68	14.68	14.61	16.92	17.01	17.08	16.91
19	13.68	13.58	13.63	13.71	15.97	16.06	15.98	15.91
21	12.83	13.03	13.08	13.11	15.17	15.21	15.18	15.06
Bench Mark [b]								
Location	North corner of patio [c]				Center of top-cover of septic tank [c]			
Level, ft.	20.00				20.00			

a No data could be collected (wells dry)

b All water levels given are with respect to the respective bench marks

c Refer to the layout plan

Appendix K.2: Recorded Water Levels in Monitoring Wells in Sprinkler Irrigation Sites (Sites 3 and 4)

	Site 3				Site 4			
	Monitoring Well Location				Monitoring Well Location			
	7	8	9	10	7	8	9	10
Trip No.	Water Surface Elevations							
2/1 [c]	a	a	a	a	a	a	a	A
4/3	a	12.83	12.54	13.14	16.40	17.65	16.59	16.40
6/5	13.95	14.13	14.54	14.34	19.55	19.55	20.49	20.25
8/7	15.35	15.88	16.99	16.49	19.55	19.55	20.49	20.15
10/9	17.10	16.88	17.54	16.94	18.50	18.60	19.09	18.90
12/11	17.45	17.68	17.84	17.79	17.80	17.80	18.19	18.15
14/13	15.25	15.43	15.29	15.34	18.10	18.20	18.69	18.70
16/15	18.70	18.73	18.03	18.34	19.70	19.65	20.84	20.70
18/17	16.85	16.63	16.24	16.54	18.80	18.80	19.99	19.50
20/19	15.15	14.73	14.24	14.64	16.65	16.60	17.39	17.30
/21	No trip				15.50	15.30	15.99	16.00
Bench Mark [b]								
Location	Center of cover of aeration tank [d]				Center of cover of aeration tank [d]			
Level, ft.	20.00				20.00			

a No data could be collected (wells dry)

b All water levels given are with respect to the respective bench marks

c Trip no. for site 3/ trip no. for site 4

d Refer to the layout plan

Appendix K.3: Recorded Water Levels in Monitoring Wells in Sprinkler Irrigation Sites

(Sites 5 and 6)

	Site 5				Site 6			
	Monitoring Well Location				Monitoring Well Location			
	7	8	9	10	7	8	9	10
Trip No.	Water Surface Elevations							
2	a	a	a	a	a	a	a	a
4	14.66	16.20	17.65	a	16.87	16.56	17.84	a
6	15.46	16.85	18.45	a	18.52	18.61	19.14	18.48
8	15.46	17.70	19.20	a	19.47	19.86	19.64	19.18
10	15.46	16.65	17.50	16.48	15.87	16.66	16.59	15.63
12	15.46	17.45	19.15	18.08	19.57	20.01	19.59	18.88
14	15.41	16.45	17.45	16.48	17.87	17.96	17.99	17.08
16	15.46	18.00	19.95	18.08	19.42	20.06	19.54	18.83
18	15.51	17.35	18.40	16.98	19.32	19.76	19.44	18.43
20	14.36	15.90	17.25	14.53	17.12	17.26	17.49	16.68
Bench Mark [b]								
Location	South-east corner of patio lab [c]				Center of cover of aeration tank [c]			
Level, ft.	20.00				20.00			

a No data could be collected (wells dry)

b All water levels given are with respect to the respective bench marks

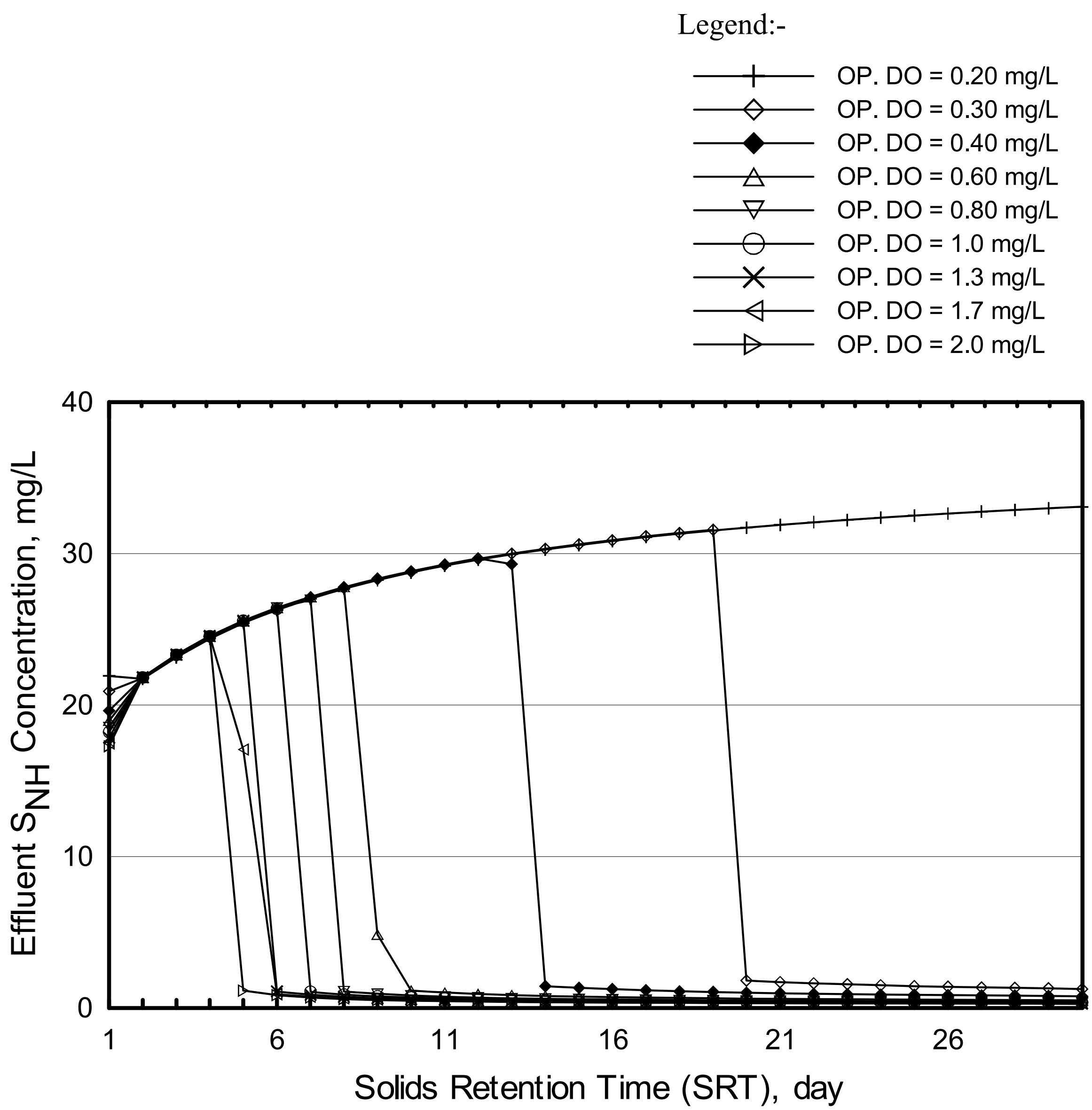
c Refer to the layout plan

APPENDIX L

GRAPHS FOR IDENTIFYING OPERATION CONDITIONS FOR SIMULTANEOUS NITRIFICATION-DENITRIFICATION PROCESS

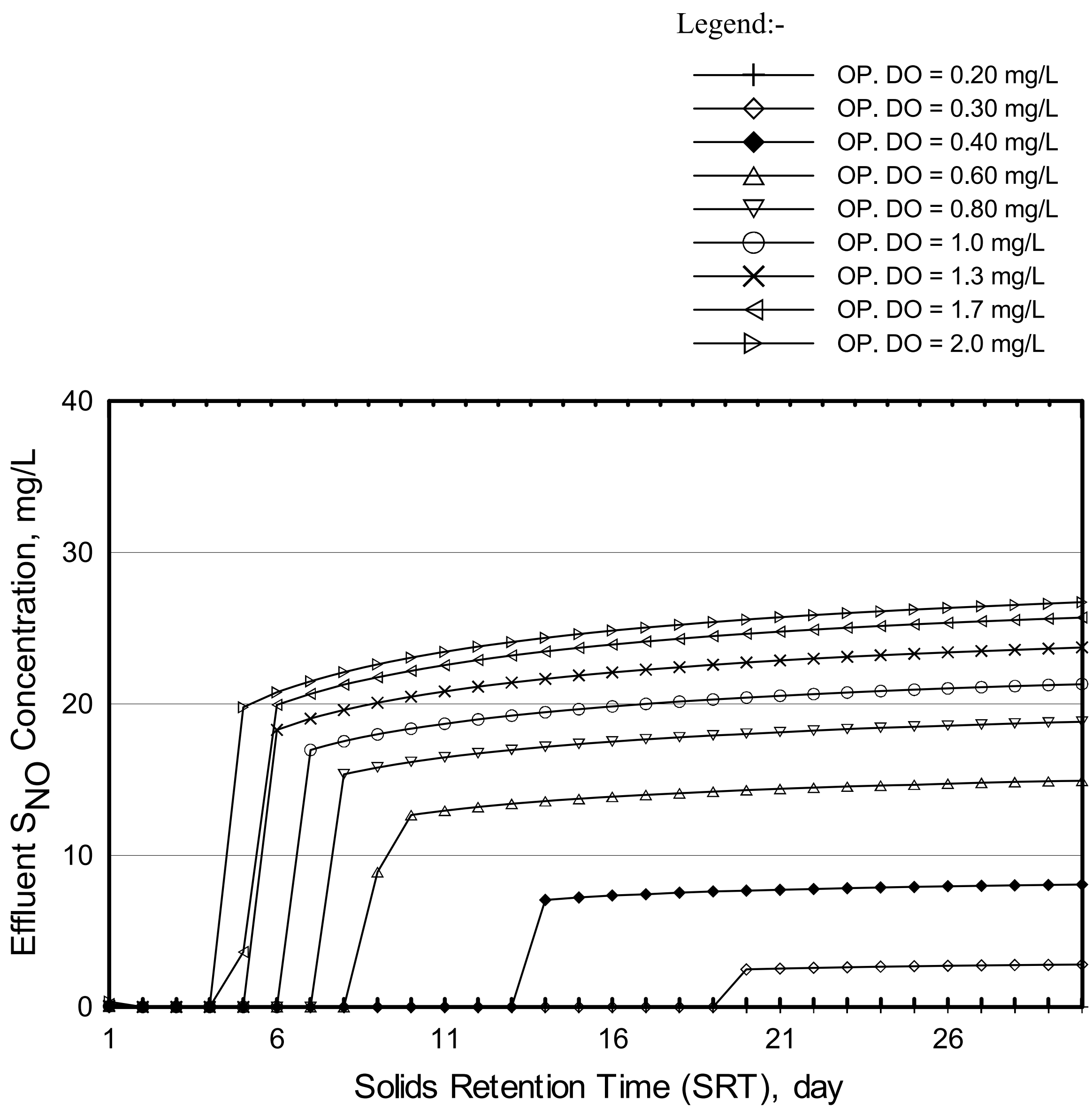
Appendix L.1

Effect of Solids Retention Time (SRT) on Effluent Ammonia-Nitrogen (S_{NH}) for varying Operating DO Concentration in a SND CSTR System



Appendix L.2

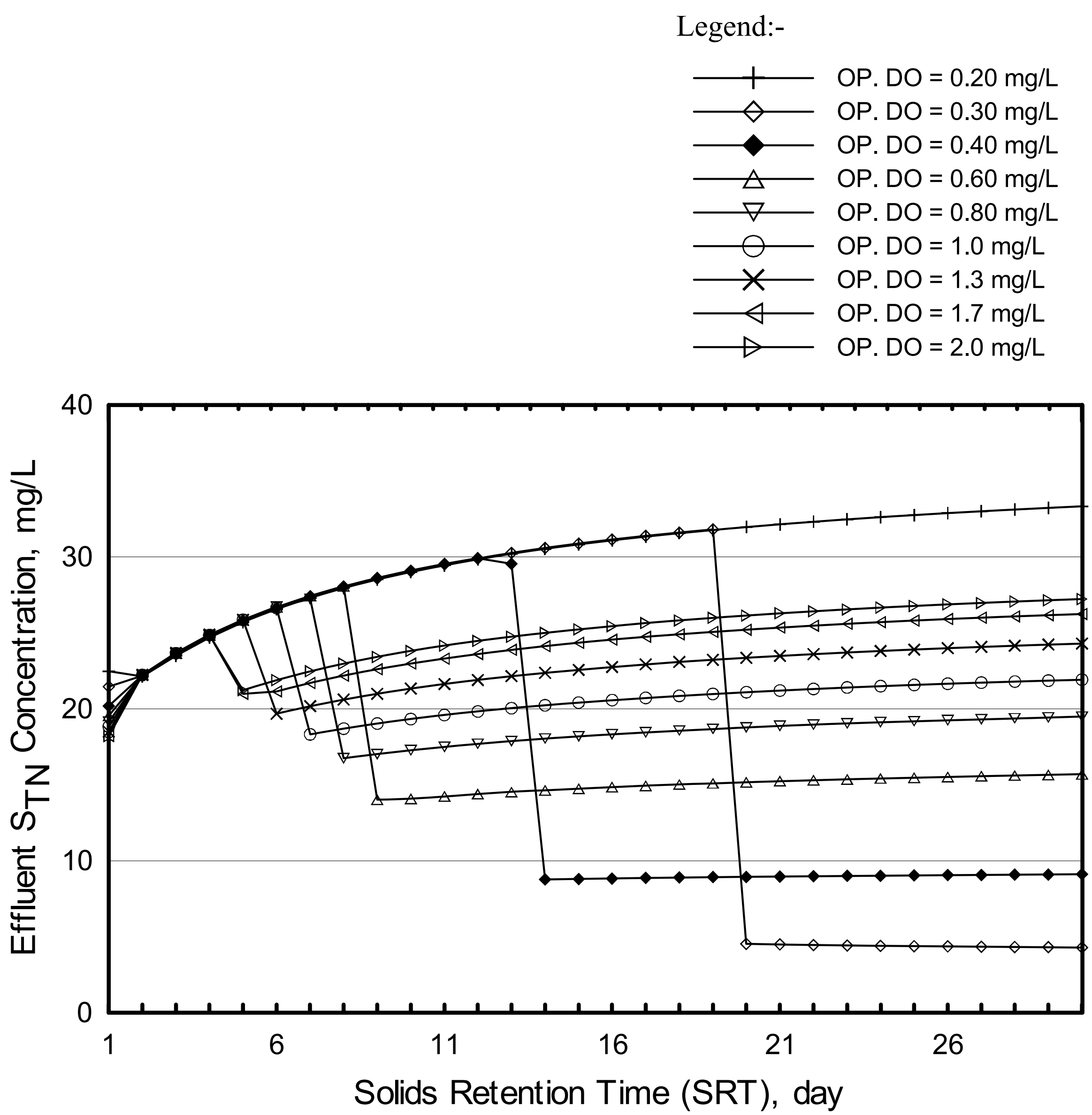
Effect of Solids Retention Time (SRT) on Effluent Nitrate (inorganic) Nitrogen (S_{NO}) for varying Operating DO Concentration in a SND CSTR System



Appendix L.3

Effect of Solids Retention Time (SRT) on Effluent Total Nitrogen (S_{TN}) for varying

Operating DO Concentration in a SND CSTR System [a]

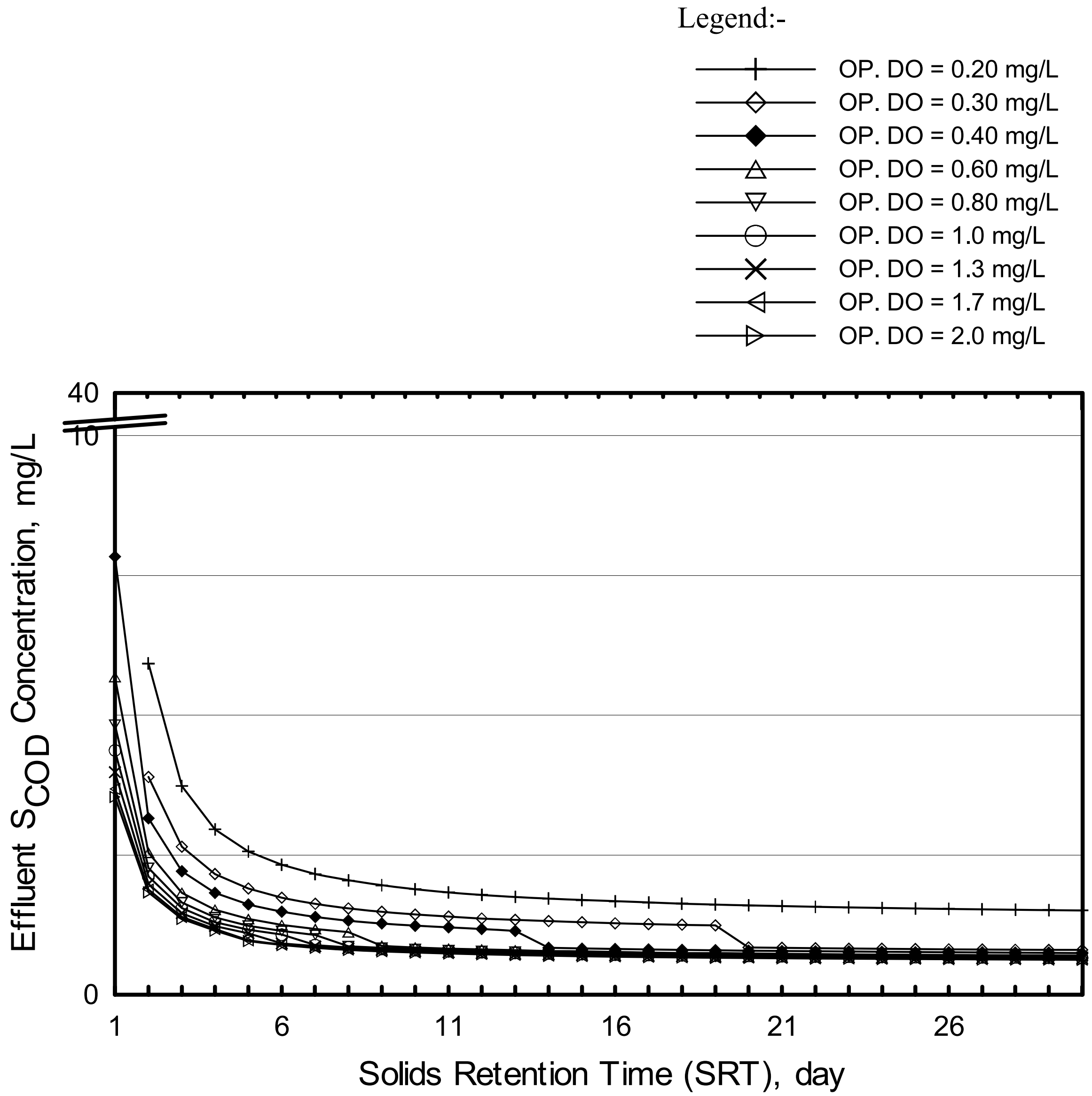


[a] Influent S_{TN} Concentration is 40 mg/L

Appendix L.4

Effect of Solids Retention Time (SRT) on Effluent Soluble COD (S_S) for varying

Operating DO Concentration in a SND CSTR System [a]



[a] Total Influent COD Concentration (S_S and X_S) is 400 mg/L

APPENDIX M

SPEARMAN CORRELATION COEFFICIENTS (SCC) AND P VALUES FOR EVALUATING THE EFFECT OF SELECTED ASM1 PARAMETERS

Appendix M.1

Spearman Correlation Coefficients (SCC) and P values for evaluating the effect of selected ASM1 parameters on S_{NH}, S_{NO}, and S_{TN} removal

		Effluent S_{NH}		Effluent S_{NO}		Effluent S_{TN}	
		SCC	P value	SCC	P value	SCC	P value
Heterotrophic parameters	Y_H	-0.0294	0.3535	0.0091	0.7751	-0.0730	0.0210
	μ_H	-0.0614	0.0524	-0.0499	0.1152	-0.1152	0.0003
	K_S	0.0006	0.9841	0.0262	0.4075	0.0266	0.4006
	b_H	0.0570	0.0781	0.0778	0.0139	0.1449	<0.0001
	K_{NO}	0.0178	0.5742	0.0358	0.2588	0.0196	0.5363
	$K_{O,H}$	-0.0546	0.0844	-0.1265	<0.0001	-0.2039	<0.0001
	η_g	-0.0108	0.7341	-0.0978	0.0020	-0.0936	0.0030
Autotrophic parameters	Y_A	0.0836	0.0082	-0.1034	0.0011	0.0800	0.0114
	μ_A	-0.3080	<0.0001	0.2589	<0.0001	-0.2435	<0.0001
	b_A	0.0782	0.0133	-0.0757	0.0166	0.0952	0.0026
	K_{NH}	0.0857	0.0067	0.0116	0.7150	0.0278	0.3794
	$K_{O,A}$	0.3279	<0.0001	-0.3090	<0.0001	0.2672	<0.0001
Hydrolysis parameters	k_h	0.0395	0.2122	-0.0369	0.2438	0.0385	0.2237
	K_X	-0.0075	0.8139	-0.0145	0.6463	-0.0142	0.6544
	η_h	0.0255	0.4212	-0.0339	0.2841	0.0047	0.8815

Appendix M.2

Identified parameters affecting S_{NH}, S_{NO}, and S_{TN} removal

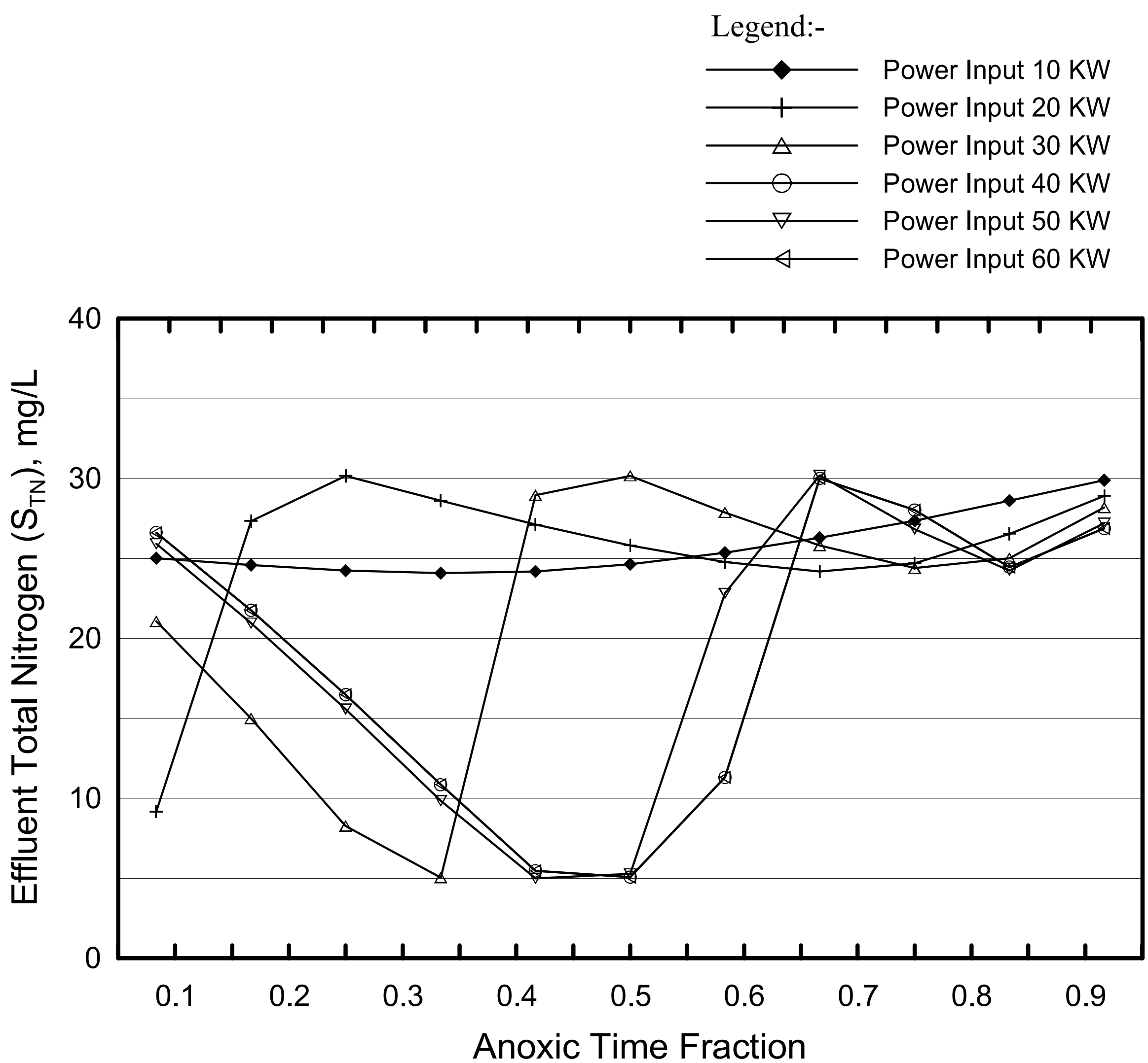
	Correlated Parameters		Most Significantly Correlated	
	Positively	Negatively	Positively	Negatively
Effluent S_{NH}	Y_A, b_A, K_{NH}, $K_{O,A}$,	μ_A	$K_{O,A}$	μ_A
Effluent S_{NO}	b_H, $K_{O,H}$, η_g, Y_A, μ_A	Y_A, b_A, $K_{O,A}$	μ_A	$K_{O,A}$
Effluent S_{TN}	b_H, Y_A, b_A, $K_{O,A}$	Y_H, μ_H, $K_{O,H}$, η_g, μ_A	$K_{O,A}$	μ_A

APPENDIX N

EFFECT OF AERATION ON NITROGEN REMOVAL IN INTERMITTENT AERATION TYPE ACTIVATED SLUDGE PROCESS

Appendix N.1

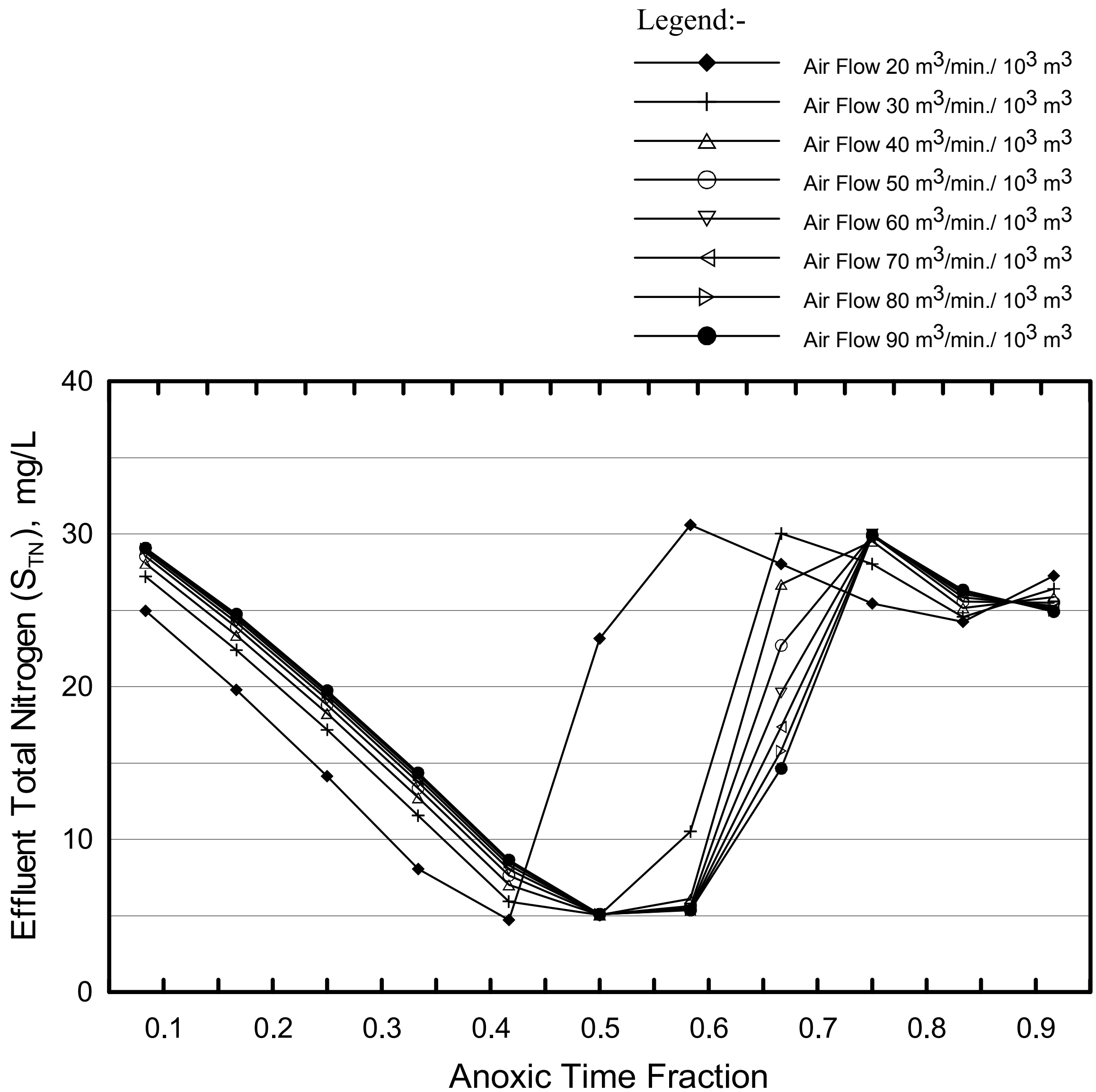
Effect of Power Input (in kW/10^3 m^3) on Nitrogen Removal for varying Anoxic Time in an Intermittent Aeration Type ASP System [a]



[a] Influent Total Nitrogen S_{TN} Concentration is 40 mg/L, Adopted Solids Retention Time (SRT) is 15-day, and Anoxic-Aerobic Cycle Time (CT) is 3-hr

Appendix N.1

Effect of Air Flow (in m^3/min/10^3 m^3) on Nitrogen Removal for varying Anoxic Time in an Intermittent Aeration Type ASP System [a]



[a] Influent Total Nitrogen S_{TN} Concentration is 40 mg/L, Adopted Solids Retention Time (SRT) is 15-day, and Anoxic-Aerobic Cycle Time (CT) is 3-hr

APPENDIX O

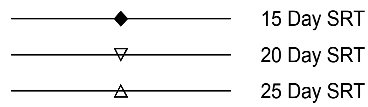
EFFECT OF CYCLE TIME ON NITROGEN REMOVAL IN INTERMITTENT AERATION TYPE PROCESS

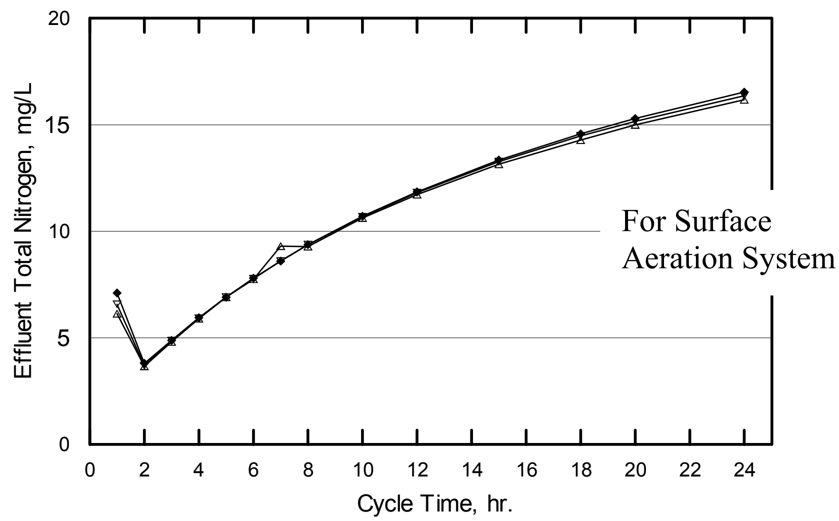
Appendix O

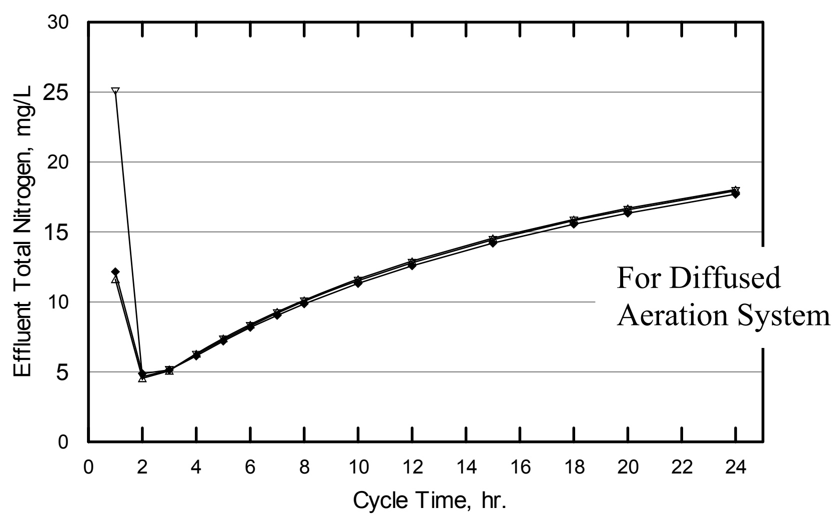
Effect of Total Cycle Time (CT) on Nitrogen Removal for varying SRTs

in a Intermittent Aeration Type ASP System [a]

Legend:-







[a] Influent Total Nitrogen S_{TN} concentration is 40 mg/L, adopted Solids Retention Time (SRT) is 15-day with selected aeration rate.