-Dissertation-

GROUND WATER MONITORING STRATEGIES TO SUPPORT COMMUNITY MANAGEMENT OF ON-SITE HOME SEWAGE DISPOSAL SYSTEMS

Submitted by

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WE HEREBY RECOMMEND THAT THE DISSERTATION PREPARED UNDER OUR SUPERVISION BY James D. Nelson

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ABSTRACT

GROUND WATER MONITORING STRATEGIES TO SUPPORT
COMMUNITY MANAGEMENT OF ON-SITE HOME SEWAGE DISPOSAL SYSTEMS

Organizations which provide centralized management of on-site sewage disposal systems are currently evolving in many areas of the United States. At present, there is no routine feedback for management agencies regarding their efforts in preventing ground water contamination. Ground water monitoring is necessary to provide these agencies with the information they need to maintain ground water quality. This research is directed toward supplying monitoring strategies for that purpose.

Ground water monitoring strategies presented herein were developed by combining deterministic and probabilistic approaches. The variables which need to be considered in such a monitoring program are discussed. Two types of monitoring are defined which need to be incorporated into an overall management plan. Inspection monitoring refers to the monitoring of individual systems to determine if they are exceeding their design discharge of pollutants to the ground water. Trend monitoring is defined as the monitoring which detects over time and space the cumulative effect of a management agencies pollution control efforts.

Inspection monitoring strategies are developed in terms of the sampling frequencies necessary to obtain a specific probability of detecting system failure. A mathematical model describing the flow of pollutants through the leach field is developed for the purpose of evaluating sampling plans and a sensitivity analysis is performed to determine the effect of varying the parameters of the system and the input to the leach field.

The discussion of trend monitoring strategies is based primarily on existing statistical theory. Sampling frequency is discussed in terms of the number of samples required to obtain an estimate of the mean of a water quality variable within specified confidence limits. The effect of spatial and serial correlation is also considered. Finally, a discussion of various sampling techniques applicable to ground water monitoring is presented.

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LIST OF SYMBOLS

c_i = cost per sample in stratum i

c_o = overhead cost of sampling

C = contaminant concentration, mg/ℓ
= total cost of sampling

C_e = concentration of treatment system effluent (determined at the water table), mg/ℓ

C_g = concentration of the ground water after mixing with the effluent, mg/ℓ

C_i = concentration of the ground water before mixing, mg/ℓ

C_o = input concentration to the model, mg/ℓ

C_t = concentration at time t, mg/ℓ

CV = coefficient of variation

D = dispersion coefficient, cm^2/day

f_i = sampling fraction for the i^{th} stratum

G_i = Fisher's measure of skewness

K = hydraulic conductivity, cm/day

L = number of strata

n = number of observations
= number of simulation runs

n_1 = number of observations in first sample

n_e = effective number of uncorrelated stations

n_i = number of observations in stratum i

n_s = number of stations

N = total number of units in the population

N_s = number of observations at each station

$\hat{p}$ = estimated value of the percent failure detection

P = precision, difference between true population mean and sample mean

LIST OF SYMBOLS
(continued)

q = Darcy velocity, cm/day

Q_e = rate of flow of effluent into the ground water, m^2/day

Q_i = rate of ground water flow, m^2/day

$\bar{r}$ = sample estimate of $\bar{\rho}$

s = sample standard deviation

s_1 = standard deviation estimated from first sample

S = source or sink term in convective-dispersion equation

t = time, days

$t_{\alpha/2}$ = value of student's t distribution corresponding to a probability of $\alpha/2$

V = seepage velocity, cm/day

V_{srs} = variance obtained from simple random sampling

V_{st} = variance obtained from stratified random sampling

V_{sy} = variance obtained from systematic sampling

$Var(\bar{x})$ = variance of the sample mean

w_i = weighting factor used in stratified random sampling

$\bar{x}$ = sample mean

$\bar{x}_1$ = estimate of the mean obtained from the first sample

$\bar{x}_b$ = biased estimate of the population mean

x_i = i^{th} observation

$\bar{X}$ = estimate of the regional mean

$\bar{X}_{st}$ = regional sample mean obtained by stratified sampling

Y_t = value of the deterministic component of a time series at time t

Z = vertical distance of contaminant travel through the leach field, cm

$Z_{\alpha/2}$ = standard normal deviate corresponding to a probability of $\alpha/2$

LIST OF SYMBOLS
(continued)

Z_t = value of the stochastic component of a time series at time t

ΔC_{o_i} = i^{th} change in inflow concentration, mg/ℓ

η = dimensionless parameter, D/VZ

θ = volumetric water content

μ = true population mean

$\bar{\rho}$ = the average of all cross-correlations from n_s stations

$\rho(k)$ = lag-k autocorrelation coefficient

σ = population standard deviation

σ^2 = population variance

σ_z^2 = $Var[z_t]$

CHAPTER 1

INTRODUCTION

Community Management Concept

Organizations which provide centralized management of on-site home sewage disposal systems are currently evolving in many areas of the United States. These management organizations are designed to provide professional levels of operation and maintenance for on-site systems within a community. The organizations which have been developed to date have no set pattern of operation or levels of activities, but rather are geared to local community needs.

Community management of on-site home sewage disposal systems is evolving into an effective means of controlling water pollution from previously poorly maintained on-site systems. In many situations the community management concept is being considered as a more cost-effective wastewater management alternative to more traditional centralized systems. Prior to implementation of this type of approach, communities in unsewered areas were frequently faced with waste management projects which had extremely high per capita operating costs.

Reasons for Increased Use of On-site Systems and Community Management

During the past few years there has been an increase in the use of on-site systems. This increased use has been caused by several trends operating in the United States. First, affluence and energy development have brought about three general types of development taking place today which lend themselves to the use of on-site systems.

1. Recreation and second-home communities. This form of development, besides often being on large lots, occurs at such slow

rates that central systems are not practical, leaving individual systems as the only alternative.

2. Bedroom communities. This form of development on the outskirts of large cities is often designed to recapture some of the rural aesthetics lost by living closer to the city; thus, large lots with individual services are common.

3. Energy boom-town communities. These communities develop rapidly during construction and often then contract once construction is completed, thus precluding large investments in central systems until more stable conditions are achieved.

Another factor which enters into the increased use of on-site systems is a reversal in attitudes toward the acceptability of these systems in providing adequate treatment. Recent research (ASAE, 1974; ASAE, 1977) has shown that, with adequate planning and maintenance, these systems can function quite well in development areas such as those described above.

Lastly, the Clean Water Act of 1977 (Public Law 95-217) has provided incentive for the use of on-site systems through the EPA construction grant program. Rhett (1977) notes that while federal construction grants have traditionally been limited to conventional centralized treatment systems, current trends are toward a more complete evaluation of the cost-effectiveness of alternative treatment methods, including on-site systems. Section 14 of the Clean Water Act of 1977 authorizes grants for privately-owned systems serving one or more principal residences or commercial establishments. As indicated by Brooks (1978) small publicly-owned treatment works, which could include on-site systems, are also grant eligible.

The entities (institutions) which could legally provide central management of on-site systems vary from state to state. A number of possible entities have been listed and discussed in a report from the Small Scale Waste Management Project (1978). Among those listed are municipalities, counties and townships, special purpose districts, private non-profit corporations, rural electric cooperatives, and private profit-making businesses.

The management organizations currently in existence exhibit a wide variance in functions and responsibilities ranging from very limited control to complete ownership of the systems within the district. This broad range of functions is exemplified by a brief review of some of the central management agencies presently operating. A more complete description of the following programs as well as several others is given by Otis and Stewart (1976) and Roy F. Weston, Inc. (1979).

1. <u>Santa Cruz County Septic Tank Maintenance District, California (Otis and Stewart, 1976)</u>. The primary function of this agency is the inspection and pumping of all septic systems within the district. The agency does not perform any monitoring nor does it design systems. As a result of the limited functions, there is little control over the effectiveness of the systems it tries to maintain.
2. <u>Stinson Beach County Water District (SBCWD), California (Wilson, et al., 1979; Roy F. Weston, Inc., 1979)</u>. The SBCWD was initially formed to evaluate the cost-effectiveness of continued use of on-site systems relative to a centralized collection and treatment system. The continued use of on-site systems was selected as the best alternative. However, it was

also decided that the SBCWD should serve as a central management agency and that a sampling and inspection program should be instituted to monitor system performance. The SBCWD management program consists of an inspection procedure wherein a wastewater technician records septic tank dimensions, tank conditions and construction, scum thickness and sludge level, and household size. Drainage conditions are also checked. If the system does not appear to be functioning properly upon initial investigation, a more extensive inspection is performed to determine the cause of failure.

3. On-Site Wastewater Management Program, Fairfax County, Virginia (Roy F. Weston, Inc., 1979). Management of on-site systems in Fairfax County, Virginia, is characterized by interaction among public agencies and private developers, lenders, and homeowners. The program is administered by the county health department and is an example of management through site evaluation and system design. Site evaluation is based primarily on soil suitability. Satisfactory construction is determined by health department inspections during the entire construction period. Operation and maintenance of the systems are the responsibility of each individual homeowner. The county's involvement in on-site systems diminishes significantly once the system begins operation. No monitoring is conducted after a system has been installed.

4. Georgetown Divide Public Utility District (GDPUD), El Dorado County, California (Otis and Stewart, 1976). This particular management agency is authorized to perform a number of

functions. Site specific systems are designed for problem areas. All systems are monitored during installation and inspected and maintained after installation. The district also has the authority to monitor water quality for the purpose of determining the effectiveness of individual systems. The GDPUD borders on complete central management of non-central systems. The only limitation is that the district does not own the systems.

From the above descriptions of currently operating management programs, it is obvious that their objectives and methods of achieving those objectives vary considerably. However, it is noted that community management of on-site systems is becoming more and more widely accepted as a viable method of dealing with waste disposal.

Increased interest in centralized management of on-site systems is also reflected in other literature concerning innovative and alternative technology. Hais, et al., (1979) reviews the current status of EPA's Innovative and Alternative Technology Program of which individual on-site systems are playing an increasingly important role. Ciotoli, et al., (1979) notes in the following quote that policies regarding on-site systems have changed drastically over the last few years at the national, state, and local level.

> "At the national level, these changes are reflected in the 1977 Clean Water Act Amendments, in the most recent EPA memorandum (PRM 79-8), and in the newly formed 'White House Rural Water and Sewer Initiatives'. At the state level, state agency organizational changes, numerous state-sponsored workshops and conferences on noncentral waste management issues, and changes in state sewerage construction grant allocation policies have taken place which have helped to alter and refocus state responsibilities toward the management of on-site and small

> community systems. At the local level, the responsibility for managing, operating, and regulating on-site and small community systems has undergone equally dramatic changes over the past several years, with management responsibilities shifting from state to local levels, and new specialized wastewater management agencies and service districts being created."

Data Requirements of Management Agencies

In order for a management entity to provide adequate maintenance of the on-site systems within its jurisdiction, there are certain data requirements which must be met. These requirements include information which is readily available such as household size, septic tank and leach field locations, and septic tank density. Also, data regarding physical characteristics of the area are important. This includes soil type, ground water levels, and ground water flow rates. Although this information will be more difficult to obtain, it will be vital to successful ground water quality management. Lastly, data on the actual concentration of contaminants in the aquifer will be necessary to quantify the effectiveness of a management organization with respect to pollution abatement. It is toward this data requirement that the research described herein is directed.

Many of the above requirements are being adequately dealt with by management agencies. The extent to which this need has been met depends to some degree on the scope of the agency's authority. An entity which has been delegated the authority for complete management of the systems within its district can generally respond adequately to data requirements necessary for site selection, alternative system design for problem areas, and detection of malfunctioning systems which result in surface failure.

However, most community management organizations do not routinely receive any feedback from the system relative to the success of the routine maintenance action with regard to deterioration of ground water quality.

Most management agencies recognize the importance of collecting data relative to minimizing the possibility of ground water contamination. Proponents of the community management concept invariably agree that some type of ground water monitoring is necessary to evaluate the effectiveness of the management organizations' activities; and the guidelines established by existing management agencies usually indicate that a monitoring program will be undertaken. However, the type of monitoring and the specific monitoring procedures are seldom delineated. For example, the following quote from the Ordinance establishing rates and charges for sewage disposal service and providing procedures for its enforcement by the GDPUD is typical (El Dorado County Health Department, 1972).

> "To assure protection of surface and subsurface waters the district will maintain a watershed monitoring program through said areas of said improvement Districts A and B, such program to be in conference with standards determined in conjunction with the (El Dorado) County Health Department, the Regional Water Quality Control Board and the Bureau of Reclamation."

The monitoring system established by the GDPUD consists of seven monitoring stations, inspection of septic tank effluents, and analysis of soil cores. In areas where system failure results in surface seepage or wastewater backing up into the house, this monitoring may be sufficient. However, a more extensive and carefully designed monitoring program will be necessary to detect degradation of the ground water quality.

Objectives

There is a need to develop ground water monitoring procedures to assist community management organizations in meeting the data requirements necessary to effectively maintain ground water quality within their jurisdictions. The purpose of this study is to provide ground water monitoring strategies which can be used to increase management effectiveness. More specifically, the objectives of this study are to:

1. Delineate the variables which should be included in a ground water monitoring program for community management of on-site sewage disposal systems.
2. Determine the sampling frequencies necessary to adequately monitor an individual on-site system.
3. Discuss statistical considerations involved in detecting ground water quality variations in an entire ground water basin as a result of input from several on-site systems.
4. Describe sampling techniques which can be used in a ground water monitoring situation and discuss relative advantages and disadvantages of each.

Scope

Two types of monitoring have been identified which will need to be incorporated into an overall community management scheme. For the purpose of this research, they are defined as follows:

1. Inspection monitoring--that monitoring involved in the routine inspection of an on-site system to determine if it is exceeding the design discharge of pollutants to the ground water.

2. Trend monitoring--that monitoring which detects over time and space the cumulative effect of the management district's pollution control efforts.

In any monitoring program it is necessary to define the type of failure the monitoring is intended to detect. Certain types of failure have been specified by established community management organizations. These failures include surface seepage of insufficiently treated effluent and back-up of wastewater into the house. Abatement of these types of failure is certainly an important function of a management organization; however, observations of this nature do not require the type of analysis followed in this study.

An equally important function of a management organization is the detection of any detrimental effects the septic tank leach field output might have on the ground water. This research effort is directed toward this type of failure.

In this study the primary emphasis has been placed on selecting sampling frequencies for both inspection and trend monitoring. However, some discussion is also given regarding sampling location for both types of monitoring.

Organization of Report

The remainder of this study is organized as follows: Chapter 2 contains a description of the variables which should be included in a monitoring program to support community management. Although several variables which can be considered important for ground water monitoring are discussed, design of the inspection monitoring program is based on the detection of nitrates alone. Rationale for this approach is also discussed in Chapter 2.

Chapters 3 and 4 involve the development of inspection monitoring strategies. The design for inspection monitoring was developed on the basis of a mathematical model. The flow of nitrates through the leach field was modeled via the convective-dispersion equation and the input to the leach field was simulated over a specified time period. Various sampling frequencies were superimposed over the output from the leach field in order to evaluate their effectiveness. Chapter 3 describes the development of the model while Chapter 4 discusses the results obtained.

Chapter 5 deals with the development of a trend monitoring program. The development of trend monitoring strategies did not involve any mathematical modeling. Rather, the statistical considerations which need to be taken into account are discussed. Also, sampling techniques which will be useful in ground water monitoring are delineated and the advantages and disadvantages of the techniques are discussed relative to ground water monitoring.

Chapter 6 concludes the study with a brief summary and a discussion of the applications to community management of on-site systems.

Information Utilization

The primary objective of this research is the development of statistical guidelines which can be employed by a management organization to supply data relative to their needs. However, it should not be overlooked that the ultimate goal of management is the transformation of this data into information which can be utilized in making decisions. This transformation from the data analysis activity to the information utilization is necessarily very subjective and is ultimately the responsibility of the decision maker.

The importance of considering information phases of regulatory water quality monitoring activities and the reluctance of most investigators to incorporate this in their studies has been discussed by Ward (1979). Acknowledgement of the fact that information utilization is of a highly subjective nature and that the worth of data obtained from any monitoring network is dependent on such factors as economic constraints and the overall objectives of monitoring as defined by the decision maker is an important consideration in the ultimate use of this report.

CHAPTER 2

SELECTION OF VARIABLES

One of the first steps in the development of any monitoring program is the selection of variables to be monitored. The problem of selecting the variables required for monitoring on-site systems is addressed in this chapter. The discussion presented has a two-fold purpose:

1. To prove background information on the variables that will be important for a management organization to monitor, and
2. To describe the advantages and disadvantages of using the different variables in a model used to develop inspection monitoring strategies.

The variables to be monitored should be representative of the particular monitoring purpose under consideration. A number of variables are important in the detection of ground water contamination from on-site systems. Two types of contamination are of primary concern--the contamination which presents a possible health hazard and the contamination which results in excessive nutrient loads leading to accelerated eutrophication of nearby surface waters.

Health Hazard Contamination

The contaminants from septic-tank effluents which can be considered important from a health standpoint are bacteria and viruses. Also, high concentrations of nitrate can be a health hazard. (Concentrations in excess of 10 mg/ℓ NO_3-N have been linked to cases of methemoglobinemia in infants). However, since much smaller concentrations can cause eutrophication problems, discussion of this variable will be deferred to the section on nutrient contamination.

Since one of the primary objectives of a septic tank leach field system is the removal of pathogenic organisms, the use of a bacteriological or virological variable as an indicator of system failure is obviously justified. The following review of literature involving virus and bacteria removal in soils is provided to indicate the role these variables have with regard to inspection monitoring and also to describe the limitations associated with use of them in an inspection monitoring model.

Viruses

The primary mechanism for virus removal in a soil system has been determined to be adsorption (Filmer, 1966; Drewery and Eliassen, 1968; Burge and Enkiri, 1978). Consequently, the removal of viruses from wastewater as it percolates through a leach field depends on the adsorptive capacity of the soil in that leach field. The ability of a soil to adsorb viruses cannot be exactly quantified on the basis of measured soil properties. However, general trends such as increased adsorption with increasing clay content, silt content, and cation exchange capacity have been noted (Drewery and Eliassen, 1968; Burge and Enkiri, 1978). Virus adsorption is also greatly affected by the pH of the soil-water system. Drewery and Eliassen (1968) showed that at pH values below 7.0 virus adsorption is rapid and effective to a high degree, but the effectiveness decreases considerably as the pH increases. The pH of septic tank effluent is generally on the order of 6.5 and 7.5 (Viraraghavan and Warnock, 1974).

Green and Cliver (1974) conducted experiments on virus removal in sand columns. They determined that a properly operating sand filtration system should present no hazards due to virus contamination. However, they did report certain important factors which should be considered. At low temperatures the sand is much less retentive and inactivation occurs at a much reduced rate. Also, operation under unsaturated conditions is necessary for adequate removal.

A study was undertaken by the Small Scale Waste Management Project (SSWMP) (1978) in order to provide information more specific to virus removal in on-site disposal systems. Prior to this investigation a number of important questions regarding virus removal, such as the long-term effects of wastewater application to the soil, had remained unanswered. Column studies indicated that silt loam soils are very efficient in virus removal. A reduction of not less than 10-thousand fold in a 10 cm depth for the duration of the study period of almost two years was reported. Sand was also found to be very effective but became somewhat less retentive after a few weeks of continuous loading, especially at temperatures as low as 6 to 8°C.

A standard loading of 5 cm/day, which resulted in unsaturated conditions, was used during these studies. Ten-fold hydraulic surge overloads were tested and some viruses were carried through the 60 cm sand columns under these conditions. Surge loading was not a factor with the silt loam soils since their hydraulic conductivity precluded too rapid infiltration of the effluent. Problems associated with surge loading of this type of soil in a field situation would manifest themselves in surface seepage or back-up of the wastewater into the home.

Under the worst conditions tested in the SSWMP study, well over 99 percent removal of the viruses was still maintained. Very little displacement of the adsorbed viruses was noted. The viruses were gradually inactivated after adsorption, the rate of inactivation decreasing with decreasing temperature. These results are somewhat contradictory to an earlier report (Carleson, et al., 1968) which implied a potential problem with wastewater disposal on soil since their investigation indicated that virus adsorption was a reversible process and the viruses were inactivated only temporarily. However, the study by Carleson, et al., (1968) used distilled water as the suspension medium as opposed to septic tank effluent used in the SSWMP study. This could explain some of the discrepancy between the two results since in the SSWMP study the principal adsorption surface was determined to be a floc which accumulated as a result of loading with septic tank effluent.

Bacteria

Several different kinds of bacteria could be used to evaluate the extent of pollution from an on-site system; however, a number of these bacteria are ubiquitous and would, therefore, serve poorly as indicators of contamination. Fecal coliforms and fecal streptocci are recommended for use as indicators of fecal pollution since they satisfy four important criteria (SSWMP, 1978):

1. They are always present in very high numbers in the feces of man and warm blooded animals and are not found in abundance in areas where fecal contamination is not present.
2. Their typical survival time is sufficiently long that they are likely to be present during the treatment process.
3. They can be monitored more easily than other bacteria because standard methods for their determination and pollution levels for maximum permissible concentrations have been developed.
4. They generally outnumber pathogenic bacteria and are therefore more likely to be detected.

Although the adsorption process probably accounts for some bacteria removal in soil sytems, the primary removal mechanism is filtration (Gerba, et al., 1975). After an on-site system has been in operation for some time, a biological clogging or crusted layer develops at the bottom of the trench beneath the drain lines (Magdoff and Bouma, 1974). Consideration of this crust layer is important in defining the hydraulic characteristics of the system as will be discussed later. The layer also serves an important function with regard to bacteria removal. Recent studies by Brown, et al., (1979) and Ziebell, et al., (1975) have indicated that there is a transition period from the beginning of operation of an on-site system until a biological clogging layer becomes established during which some systems, in particular sandy soils, do not function adequately in the removal of bacteria. The length of this transition period has not been quantified, but from the studies conducted it seems to be on the order of 3 to 6 months. However, this clogging layer, once established, acts as a very effective filter for bacteria.

A necessary requirement for bacteria removal, as in the case for virus removal, appears to be sufficient depth of an unsaturated zone. This unsaturated flow, induced by either a clogging zone or a limited application rate, causes a more tortuous flow path through the smaller pores of the soil matrix thereby enhancing removal by forcing more soil-water contact. Two recent studies (Reneau, 1978; Hagedorn, et al., 1978) have indicated that bacteria can move long distances in a relatively short period of time under saturated flow conditions.

Data collected from column studies by Ziebell et al. (1975) suggests that 60 cm of sand is adequate in removing fecal bacteria.

However, such factors as flow regime and temperature clearly affect the removal process and, for added safety, more than 60 cm depth should be provided. Ziebell, et al. (1975) also studied bacteria removal in silt loam columns. The results indicate that 50 cm of silt loam soil is sufficient for removal of fecal bacteria under normal flow regimes encountered in the field. However, some short circuiting through large pores or cracks did occur resulting in the bypass of bacteria through some of these columns. The authors noted that this situation was probably characteristic of the column studies only since the large pores and cracks which occurred in the columns would not extend indefinitely into the soil under field conditions.

This conclusion is supported by field data (Brown, et al, 1979). In this study movement of fecal coliforms was monitored in situ in three different types of soils. A sandy loam soil, sandy clay soil, and a clay soil were investigated. The clay soil proved most effective with very low concentration collected at any point below the clogged zone. In the sandy soil and sandy loam some isolated high concentrations were noted as a result of travel through root channels. Fecal coliforms were detected below 120 cm only occasionally. A 10- to 100-fold reduction within 5 cm was not uncommon. In most cases fecal coliforms were not present below 35 cm.

Conclusions

Studies to date seem to indicate that properly designed on-site systems are very efficient in the removal of viruses and bacteria from septic tank effluent. With proper emphasis placed on site evaluation and correct installation procedures, bacteriological or virological contamination is unlikely. This is not intended to imply that monitoring for these contaminants is unimportant given the grave consequences of allowing malfunctioning systems to continue operation. This type of monitoring may be particularly important where the leach field overlies fractured bedrock as is frequently the situation in mountainous areas. Bacteria have been shown to travel very long distances in these types of locations (Millon, 1970; Morrison and Allen, 1972; Waltz, 1972).

Although considerable research has been conducted in recent years regarding bacteria and virus removal, the modeling of the removal processes has not yet reached a point of general application as required for this study. The approach used in this research was to develop inspection monitoring strategies by mathematically modeling the flow of contaminants through the leach field. Hence, it was decided that some other variable which could be modeled more accurately would be more appropriate for this purpose than bacteria or viruses. Another reason for not including bacteria or viruses in the design of inspection monitoring was that if the system is functioning properly, bacteria and viruses should be completely removed. Detection of the occurrence of only sporadic failures would require inordinately high sampling frequencies.

A number of approaches can be used in lieu of intensive bacteriological monitoring to assure adequate purifiaction. Careful attention to site evaluation and proper installation can reduce the monitoring requirement. Also, simply monitoring the ground water level can be an effective strategy. If the depth of the unsaturated zone is sufficient, minimal sampling for fecal indicators should be adequate. In this case the sampling frequency could be the same as that required for some other variable, e.g. nitrate. However, if the ground water level increased to the point that effective bacteriological purification is limited, an increase in sampling frequency is justified. Some systems will require more intensive monitoring due to their location or the type of the soil. On-site systems located in mountainous areas should be monitored more carefully, as should systems with very sandy soils.

Nutrient Contamination

As previously indicated another type of contamination from on-site systems which is of major concern is that which results from excessive nutrient loads. The importance of this contamination depends to a large extent on the location of the system. Those which are located in areas very near bodies of surface water should be monitored carefully for nutrient contamination. Concentrations of phosphorus as low as 0.02 to 0.05 mg/ℓ and nitrate concentrations on the order of 0.5 mg/ℓ have been shown to cause eutrophic conditions in surface waters (Norvell and Frink, 1975).

Phosphorus

Research indicates that on-site disposal systems are very effective in the removal of phosphorus from wastewater. When phosphorus is applied to the soil, it is initially adsorbed to the soil mineral surfaces. As the phosphorus concentration in the soil solution increases, precipitation reactions may occur (Sikora and Corey, 1976). In a laboratory investigation of phosphorus retention Magdoff and Keeney (1976) reported 121 μg/g retained in sand columns. Based on this information and data from other studies, Sikora and Corey (1976) estimated the depth of phosphorus penetration to be approximately 50 cm/year in sandy soils and as low as 10 cm/year in finer textured soils. This would indicate the possibility of ground water contamination in sandy soils with high ground water levels. Laboratory studies from Sawhney (1977) also suggest the possibility of phosphorus contamination in areas with high or perched water tables and soils with low adsorptive capacities.

However, analysis of samples from a sandy soil around a seepage pit that had been operating for 12 years showed that phosphorus had moved a horizontal distance of only 135 cm (Sawhney and Hill, 1975). Laboratory studies revealed that the soil should have been saturated with phosphorus up to a distance of 5 meters. Yet the soil was not fully saturated even at a distance of 15 cm suggesting that the sorption sites are regenerated with time. This regeneration was confirmed in the laboratory by alternating wetting and drying phases. Thus, extrapolating laboratory results of field situations may be very misleading. This is confirmed by another study in which concentrations of only 0.5 mg/ℓ were found at a depth of 60 cm in a six year old system (Sawhney and Starr, 1977).

In a recent report by Jones and Lee (1979), results obtained from a four year study showed that on-site systems will not, in general, be a source of phosphorus for surface waters. This study involved intensive monitoring of ground water adjacent to a septic tank leach field. The study area was a recreational development located in northwestern Wisconsin which has received criticism because of the possibility of ground and surface water contamination resulting from septic tank discharge. The data collected indicated that septic tank effluent did migrate into the ground water around the region. However, phosphorus was apparently retained very effectively since none was found in any of the monitoring wells even though the soils in the area were very sandy and had a very low adsorptive capacity. The authors conclude that in a limited number of circumstances where septic tank disposal systems are located immediately adjacent to surface waters it is conceivable that phosphorus may be contributed to those waters. However, in most cases phosphorus will not be transported far enough through the soil to enter surface waters.

Results of the investigations reviewed above indicate that in most cases phosphorus contamination from on-site systems will not be a major problem. Therefore, designing a general monitoring network to account for phosphorus contamination is not recommended, although some site specific situations would certainly warrant considering phosphorus contamination as a factor in monitoring design.

Nitrogen

The predominant forms of nitrogen in septic tank effluent are ammonium and organic nitrogen. Ammonium constitutes about 75 percent of the total nitrogen content and organic nitrogen constitutes about 25 percent (Otis, et al., 1974). When effluent leaves the septic tank, further nitrogen transformations take place. Organic nitrogen is rapidly mineralized to ammonium. Ammonium in solution is readily sorbed by soil particles due to the cation exchange capacity of the soil (Lance, 1972). The ammonium ion can be leached to the ground water if anaerobic conditions exist in the leach field and if the cation exchange sites in the soil have become equilibriated with the cations in the effluent (Sikora and Corey, 1976). The ammonium ion can be fixed in a stable form by expanding clay minerals if it is trapped between the layers of these minerals, and under certain conditions ammonium reacts with soil organic matter to form complexes that are resistant to leaching and decomposition (Lance, 1975). However, it is unlikely that either of these reactions will be extensive in leach fields.

The two primary mechanisms of nitrogen transformation in a septic tank leach field are nitrification and denitrification. Nitrification refers to the biological oxidation of ammonium, first to nitrite, then to nitrate. Denitrification is the conversion of nitrate to nitrogen gas.

Nitrification is an aerobic reaction with nitrate being the predominant end product. Nitrate is considered a conservative pollutant and is therefore of major concern since it can pass easily through the soil. Nitrate can be immobilized by plant uptake and by microbial incorporation into the biomass of certain organisms, but neither of these processes can be considered significant in leach field systems.

Denitrification is the only mechanism by which enough nitrate can be removed from the septic tank effluent to be considered significant (Sikora and Keeney, 1976). Denitrification occurs in the absence of oxygen and the nitrogen must be in the nitrate form. Therefore, nitrification, an aerobic process, must occur before denitrification, an anaerobic process. This implies two completely opposite aeration requirements in the same spatial proximity for the removal of nitrogen.

Denitrification also requires some energy source. The available carbon from the wastewater is usually not sufficient. Stewart, et al., (1979) reported that in column studies, available carbon from the leachate which was usable as an energy source was completely exhausted between the 42nd and 95th day of their tests. Thereafter, nitrate reduction as the soil solution passed through the saturated zone remained below 28 percent.

Conclusions

From the above discussion it is obvious that on-site systems function poorly in the removal of nitrogen from wastewater. Operation in the unsaturated flow regime, which is necessary for adequate bacteriological and virological treatment and for phosphorus removal, also establishes conditions which allow for rapid nitrification within the first few centimeters of unsaturated flow. Simple dilution of the nitrate with the ground water provides adequate reduction of the nitrate concentrations if the density of septic tanks in a given area is sufficiently low. Calculation of the nitrogen input from septic tank systems indicates that this source would not result in nitrate-N concentrations in the ground water in excess of 10 mg/ℓ when the density of the systems is less than 1 per 0.2 hectare (Walker, et al, 1973b). However, higher densities could result in significant increases in nitrate concentrations in the ground water.

Unlike the modeling of bacteria and virus removal processes, the modeling of nitrate flow through a porous medium has been well documented. Nitrate movement through soil can be modeled accurately enough to be used for the design of inspection monitoring.

For the above reasons it seems that nitrate would serve as an important variable to monitor from a water quality management standpoint. Based on a sampling program set up to monitor nitrates, rational decisions could be made regarding the allowable density of on-site systems in a given area or when some type of alternative treatment is necessary. For the purpose of this study nitrate was the only variable which was included in the model used to study inspection monitoring design.

Other Variables

A number of other variables which have been included in previous investigations of on-site systems have not been discussed here. These include such variables as BOD, COD, total suspended solids, and chlorides. Although studying these quantities has led to increased understanding of the operation of on-site systems, there is little need to include most of these variables in a water quality management program. Considering the fact that the wastewater is ultimately being discharged into an anaerobic environment, the oxygen demand of that wastewater is unimportant. Measurements on suspended or dissolved solids could be important in terms of leach field clogging but would serve little purpose in a ground water monitoring program as described here.

Chloride measurements have frequently been used as an indication of the overall extent of pollution and as such can be an important variable in a monitoring program. However, chloride measurements will not give any indication of the degree of treatment provided by a leach field and, therefore, should not be considered as important as the bacteriological and nutrient variables previously described.

Because of the budgetary and time constraints that will be imposed on a management agency, it will be necessary to limit the variables to be monitored to those which can be considered most important in terms of meeting the monitoring objectives. Some special situations may present the need to include more variables than those described in this chapter. If this is the case the specific variables to use will be dictated by the particular problem being considered. However, for most situations involving ground water monitoring of on-site systems, the variables previously discussed will suffice.

CHAPTER 3

DEVELOPMENT OF THE MODEL

In order to evaluate the effectiveness of several proposed sampling plans for inspection monitoring, a model was developed which simulates the operation of an individual on-site system. This chapter provides a description of that model and discusses the assumptions made in the model development.

From the previous discussion on the selection of variables which need to be monitored, it follows that nitrate is the most logical variable to incorporate into the model. Two types of models were combined. A mass transport model was used to trace the actual physical movement of the pollutant through the leach field and a simulation model was used to establish the input concentrations.

The application of the model described herein is limited to the evaluation of sampling plans for inspection monitoring as previously defined. General sampling techniques for trend monitoring will be discussed in Chapter 5. It is emphasized that this model is of a very hypothetical nature and the results should not be construed as an exact quantification of the performance of an actual on-site system. Rather, they should be viewed as a relative means of comparing several sampling plans and as an indication of the importance of the various factors considered in designing an overall monitoring strategy.

Mass Transport Model Description

The flow of nitrate through the leach field was modeled via the convective-dispersion equation:

$$\frac{\partial C}{\partial t} = D \frac{\partial^2 C}{\partial Z^2} + V \frac{\partial C}{\partial Z} + S \quad (1)$$

where D is the dispersion coefficient of the contaminant, C is the contaminant concentration, t is time, Z is the distance traveled through the soil, and V is the seepage velocity of the wastewater through the media. S represents a source or sink term which accounts for changes in contaminant concentration due to reactions such as adsorption, biological uptake, or other similar mechanisms. For the purpose of this study nitrate is considered a conservative pollutant; therefore, S is assumed to be zero.

The solution of equation 1, with S = 0, is given by Ogata and Banks (1961) as

$$\frac{C}{C_o} = \frac{1}{2}\left[\operatorname{erfc}\left(\frac{Z - Vt}{2\sqrt{Dt}}\right) + \exp\left(\frac{VZ}{D}\right)\operatorname{erfc}\left(\frac{Z + Vt}{2\sqrt{Dt}}\right)\right] \quad (2)$$

where C is the concentration at time t, C_o is the input concentration, erfc is the complimentary error function and all other quantities are as previously defined. The initial and boundary conditions used to obtain the above solution are

$$C(0,t) = C_o \; ; \; t \geq 0$$

$$C(x,0) = 0 \; ; \; x \geq 0$$

$$C(\infty,t) = 0 \; ; \; t \geq 0.$$

Ogata and Banks (1961) indicate that in many cases it is only necessary to consider the first term in equation 2. They present a quantitative method by which the error associated with neglecting the second term can be determined as a function of $\eta = \frac{D}{VZ}$.

For the values of the parameters considered in this study, the largest value of η was calculated as 1.79×10^{-2}. For this value the second term in equation 2 may be ignored with a maximum error of less than 9 percent. For all other values of η calculated from the parameters used, ignoring the second term results in a maximum error of less than 1 percent. Therefore, the solution to equation 1 used in this study was taken as

$$\frac{C}{C_o} = \frac{1}{2}\left[\operatorname{erfc}\left(\frac{Z - Vt}{2\sqrt{Dt}}\right)\right]. \tag{3}$$

The variation of the input concentration with time was simulated as a step function. The concentration at the sampling point at time t was determined by superposition of equation 3. That is

$$C_t = \sum_{i=1}^{n} \frac{\Delta C_{o_i}}{2}\left[\operatorname{erfc}\left(\frac{Z - V(t - t_{i-1})}{2\sqrt{D(t - t_{i-1})}}\right)\right] \tag{4}$$

where C_t is the concentration at the time of interest t, ΔC_{o_i} is the i^{th} change in the inflow concentration, t_{i-1} is the time of the i-1 change, and n is the number of changes which have occurred prior to time t.

Equation 4 was solved for two vertical portions of the leach field--first for the unsaturated zone, and then for the saturated zone (see Figure 1). The input concentration for the unsaturated zone was taken as the concentration of the septic tank effluent. The input concentration for the saturated zone was taken as that concentration which was calculated at the water table surface.

For the purpose of this study the assumption was made that all ammonium was immediately converted to nitrate upon entering the unsaturated zone directly below the biological crusted layer. There is evidence that this transformation does take place within the first few centimeters of a well aerated soil. Walker et al. (1973a) noted that in sandy soils nitrification commenced in the unsaturated zone about 2 cm from the crusted layer and that nitrification was essentially complete

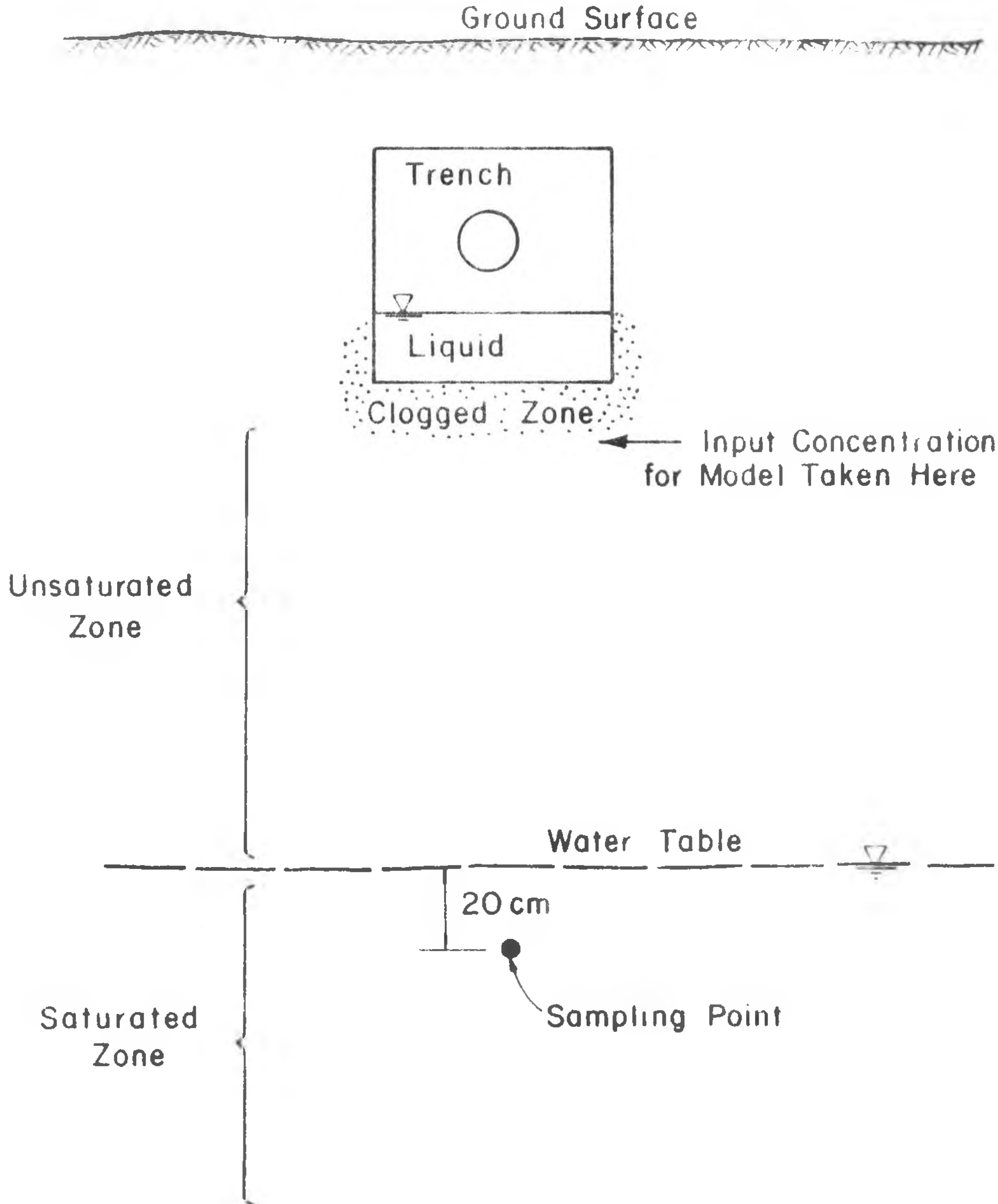

Figure 1. Cross Section of Leach Field as Assumed in the Model.

within 6 cm. Since there is almost no nitrate in septic tank effluent, the input nitrate-N concentration for the model was taken as the ammonium-N concentration of the septic tank effluent.

In order to determine the nitrate concentration of a sample at a specified time, equation 4 was evaluated at the sampling depth which was taken as 20 cm below the ground water level. The rationale for locating the sampling point in the upper portion of the saturated zone was based on the classification of the treatment system as shown in Figure 2. All portions of the system from the septic tank to the lower boundary of the unsaturated zone were considered a part of the treatment system. Inspection monitoring is designed to determine the effectiveness of the treatment system itself and thereby quantify the output quality of the system. This will, in turn, be the input to the ground water aquifer. This approach is comparable to the effluent monitoring procedures as discussed by Pöpel (1976) and Berthouex (1974). Further discussion of the parallels between inspection monitoring and effluent monitoring will be presented later.

There is some justification for not limiting the treatment system to the unsaturated zone, but including the saturated zone as well. However, in many situations, such as areas with a high density of on-site systems and mountainous areas where the leach field is located over fractured bedrock, it is necessary to insure complete treatment of the wastewater before it reaches the water table. Since this model will be used in the evaluation of inspection monitoring strategies for individual systems, it seems reasonable to locate the sampling point where treatment is assumed to be completed.

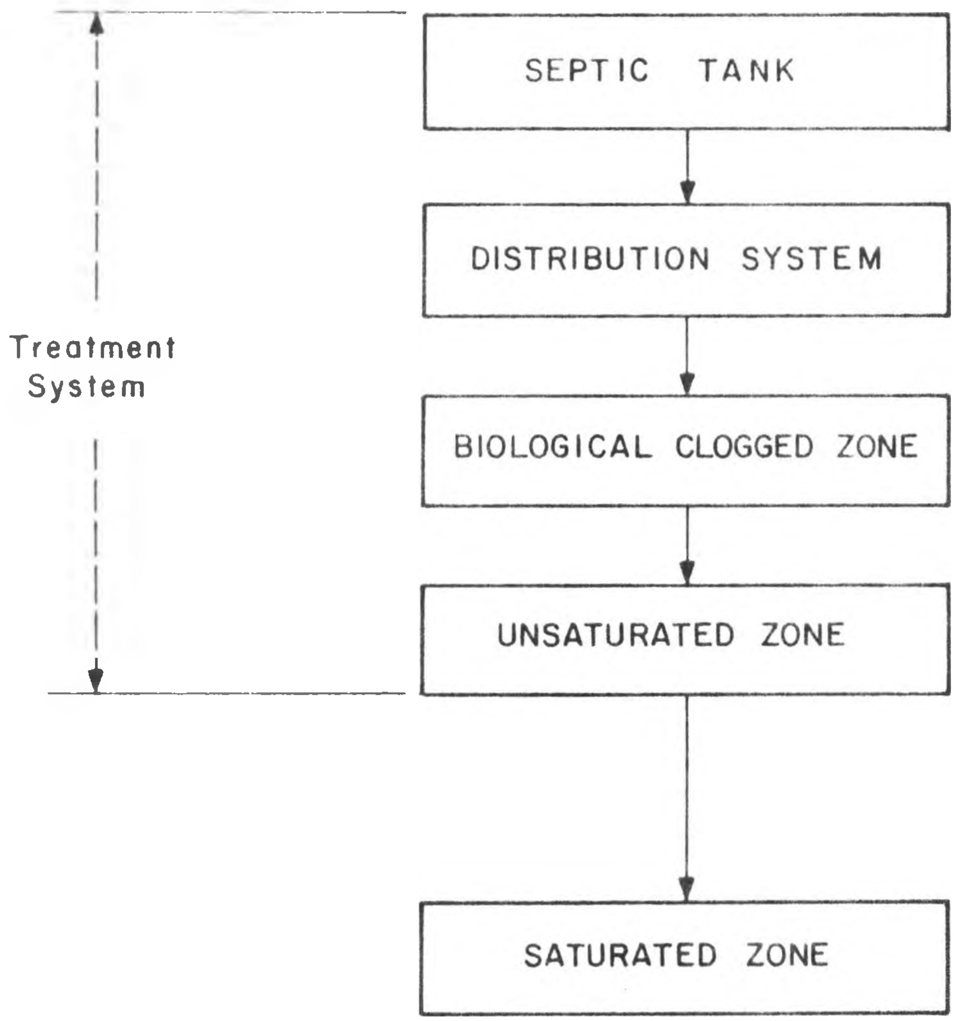

Figure 2. Flow Chart Indicating Classification of the Treatment System.

In the model the ground water level was set at 150 cm. Two other levels, 90 cm and 250 cm, were considered in the sensitivity analysis.

Determination of Transport Model Parameters

Calculation of Seepage Velocities

Seepage velocities used in equation 4 were determined on the basis of research conducted by Bouma (1975). This study quantified the effect of the biological crust layer on infiltration of septic tank effluent into the soil. Thirteen subsurface seepage systems, ranging in age from 0.5 to 12 years, were studied. The systems were divided into four general soil types: sand, sandy loam, silt loam and clay. Subcrust tension was measured in each of the systems studied and the corresponding hydraulic conductivity determined. From these data hydraulic conductivity curves were developed for each soil type as shown in Figure 3.

A sandy soil was assumed in the model developed for the present study and a silt loam soil was also considered in the sensitivity analysis. The average hydraulic conductivities for the sandy and silt loam soils studied by Bouma (1975) were utilized to determine the estimated seepage velocities used in the model. The assumption was made that the water table was deep enough for the Darcy velocity, q, to be equal to the hydraulic conductivity, K. Figure 4 was then used to determine the volumetric water content, θ, from which the seepage velocity may be obtained.

The assumption that the Darcy velocity equals the hydraulic conductivity requires that the hydraulic gradient be unity. This is only an approximation since the gradient is not constant but varies with the flow rate and hydraulic conductivity of the medium (Bouma, 1975). If

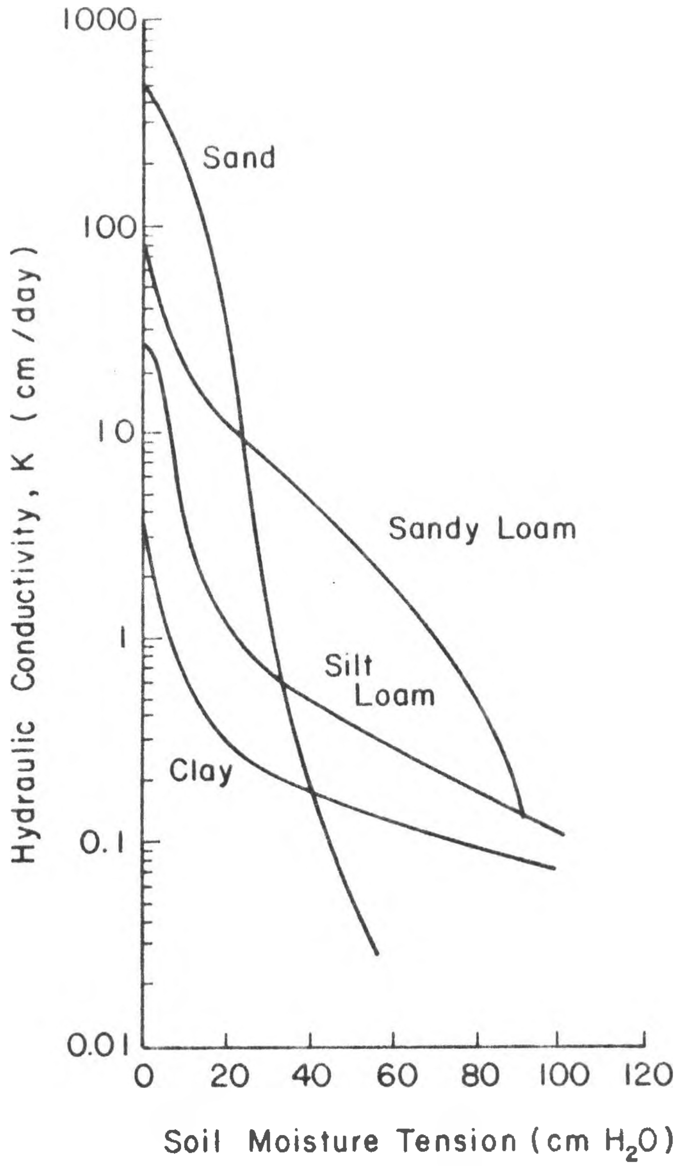

Figure 3. Hydraulic Conductivity Curves for Four Soil Types (Bouma, 1975).

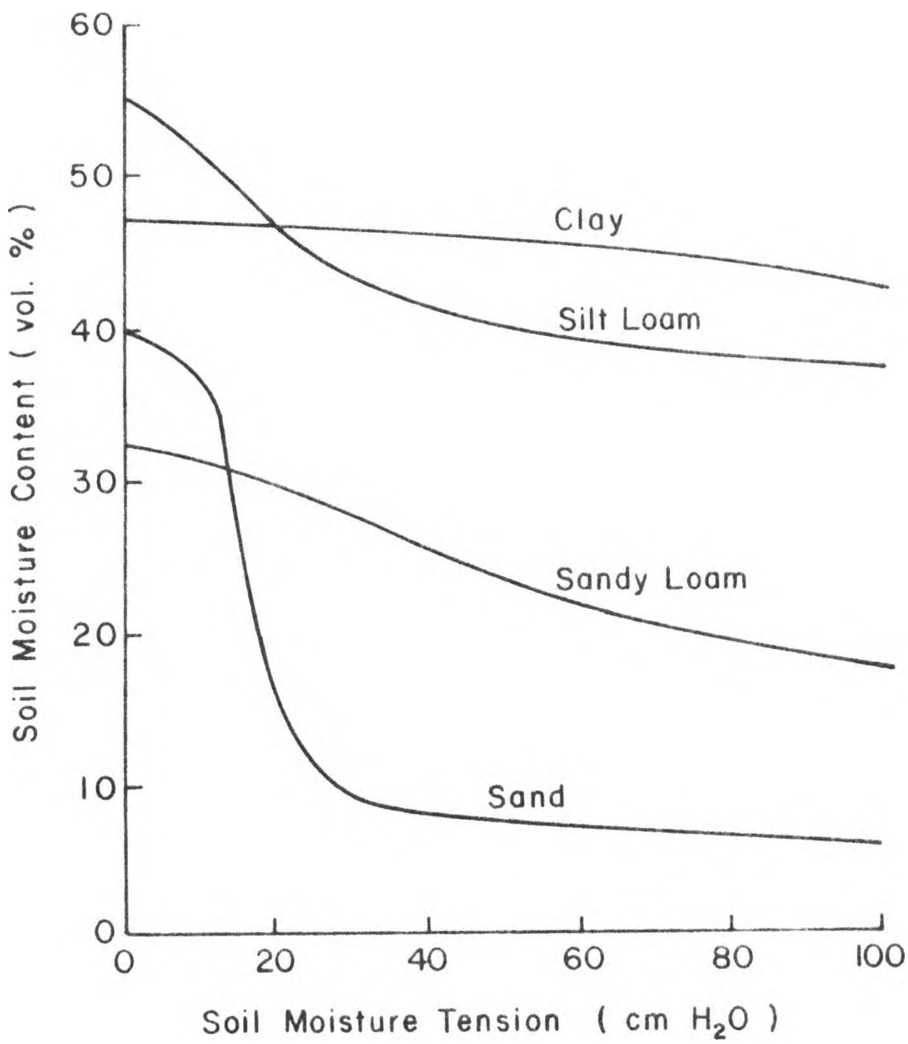

Figure 4. Soil Moisture Retention for Four Soil Types (SSWMP, 1978).

the water table is deep enough, the gradient will be close to unity down to a specific depth and will then gradually decrease from that depth to the water table. The assumption can be violated due to seasonally high or perched water tables or the development of a ground water mound beneath the leach field.

It is accepted that the estimated seepage velocities based on the above assumptions are very approximate. However, it will be shown later that the results of this study, i.e., the evaluation of sampling plans, are not very sensitive to the parameters selected for the transport model.

Four subsurface seepage systems with sandy soils were studied by Bouma (1975). The average subcrust tension for these systems was 24.2 cm and the average hydraulic conductivity, K, was calculated as 6.8 cm/day. These values were assumed for the average tension and average hydraulic conductivity in the unsaturated zone for the sandy soils considered in the model. From Figure 4 the value of the volumetric water content, θ, was taken as 0.10. This results in an average seepage velocity ($V = q/\theta$) of 68 cm/day for the unsaturated zone.

The same type of analysis was followed for determining the seepage velocity in the saturated zone. From Figure 3, K_{sat} = 500 cm/day was selected and θ_{sat} = 0.40 was taken from Figure 4. Again assuming that $K = q$ a value for the seepage velocity of 1,250 cm/day is calculated. This is obviously a much higher value than actually occurs due to the fact that in the saturated zone the hydraulic gradient will be negligible. Therefore, the Darcy velocity will be much less than the hydraulic conductivity. It is accepted that since the water table is essentially static this is at best a very gross approximation. However, the fact that only 20 cm of saturated flow is accounted for in the model makes the determination of the parameters for saturated flow much less important than those for unsaturated flow.

For sensitivity analysis the model was run assuming a silt loam soil instead of a sandy soil. The seepage velocities for this case were determined in the same manner as described above. The average subcrust tension used for the silt loam soil was 27 cm and the corresponding hydraulic conductivity was calculated as 0.8 cm/day (Bouma, 1975). From Figure 4 a value of $\theta = 0.43$ was chosen. The unsaturated seepage velocity, in turn, was calculated as 1.9 cm/day. The saturated hydraulic conductivity from Figure 3 is 25 cm/day. From Figure 4 a value of $\theta = 0.55$ is given which yields 45.5 cm/day as the saturated seepage velocity.

Calculation of Dispersion Coefficients

The last parameter which needs to be considered for the solution of equation 4 is the dispersion coefficient, D. Dispersion coefficients have been shown to be different for the same soil depending on whether the flow is saturated or unsaturated (Kirkham and Powers, 1971). An equation which was developed by Yule and Gardner (1978) for dispersion coefficients as a function of seepage velocity in unsaturated Plainfield sand was used to obtain all estimates of D for this study.

Other empirical equations for dispersion coefficients under saturated conditions are available in the literature (Ebach and White, 1958; Harleman and Rumer, 1963; Hoopes and Harleman, 1965). However, a comparison of the results obtained from these equations and the Yule-Gardner equation with the results from a field study by Misra and Mishra (1977) indicates that the Yule-Gardner equation provided more realistic dispersion coefficients even for the saturated case. Therefore, this equation was used for both unsaturated and saturated flow conditions.

The Yule-Gardner equation is given as

$$D_L = 0.216V + 0.0032$$

where D_L is the longitudinal dispersion coefficient in cm^2/min, and V is the seepage velocity in cm/min. From this equation a value of 19.3 cm^2/day was determined for the unsaturated sand and a value of 275 cm^2/day was determined for the saturated sand. Values for the silt loam were determined as 5.0 cm^2/day and 14.4 cm^2/day for the unsaturated and saturated cases, respectively.

Sensitivity analysis indicated that, in terms of the results sought in this study, the values selected for the dispersion coefficients are not extremely critical. This is particularly true for the saturated case as might be expected given the minimal distance of travel under saturated conditions considered in the model.

Simulation Model Description

A simulation model based on various random components was used in conjunction with the mass transport model discussed in the previous section. The concentrations of septic tank effluent were simulated over a one-year time period. This simulation was then used as the input to the mass transport model and the concentration with time was determined at the sampling point. There were a number of random components to the simulation so that different results were obtained on each simulation run. The output resulting from each simulation was superimposed with a sampling plan in order to determine the effectiveness of that plan.

Critical Concentration Values

Two critical values, failure concentration and detection concentration, were required for the simulation portion of the model. Failure concentration is defined as the concentration which was superimposed on the input to the mass transport model to represent a system failure. Detection concentration is that concentration above which a failure was assumed to be detected. The detection concentration was set at a smaller value than the failure concentration since the output of the mass transport model would tend to be damped due to dispersion.

For the purpose of this research, failure and detection concentrations were based on the distribution of ammonium from septic tank effluent. It is recalled that the nitrogen concentration of septic tank effluent consists almost entirely of ammonium and organic nitrogen. Assuming that ammonium is immediately converted to nitrate upon entering the leach field, determining critical concentrations on the basis of the distribution of ammonium from septic tank effluent is tantamount to determining critical concentrations based on the distribution of nitrate input to the transport model. It was also assumed that the septic tanks included in the sample from which the ammonium distribution was determined were functioning properly. This approach is analagous to the recommended procedure for establishing effluent standards (Hunter, 1977). The method involves selecting treatment plants which represent the best practical control technology currently available and determining the effluent quality distribution. Standards are then set according to the probability that a certain value will be exceeded thereby incorporating the probabilistic nature of the problem.

This concept is further illustrated in Figure 5. The line labeled μ represents the true value associated with the pollutant process as concentrations change with time. However, due to the random variability associated with the process, any observation at time t will not generally correspond exactly to μ. Instead, the observation will be at some value, say x, which is equal to the true value μ plus some random variation. A normal (Gaussian) curve superimposed on the figure at time t represents the distribution of all possible observations at that time. Assuming that this distribution is actually known, it is possible to establish critical values based on the probability of exceedence of those values.

Critical values of this study (i.e., failure and detection concentrations) were based on the assumption that the variables were normally distributed. Theoretically, the probability of exceedence of a particular value can always be determined regardless of the underlying distribution if that distribution is known. However, justification for using a more complicated distribution for the purpose of setting critical values is questionable. Hunter (1977) notes that the use of complicated distributions, accompanied by long discussions on how to estimate necessary parameters and determine probability statements, cloud the fact that simplicity often works just as well. He recommends use of the normal distribution if the data appear to be approximately normal or can be made so by a simple transformation.

Referring again to Figure 5, for the problem of setting standards the true concentration, μ, is generally considered a constant with time. For the purpose of determining the required critical values for this study, this assumption was accepted, thereby reducing the problem to

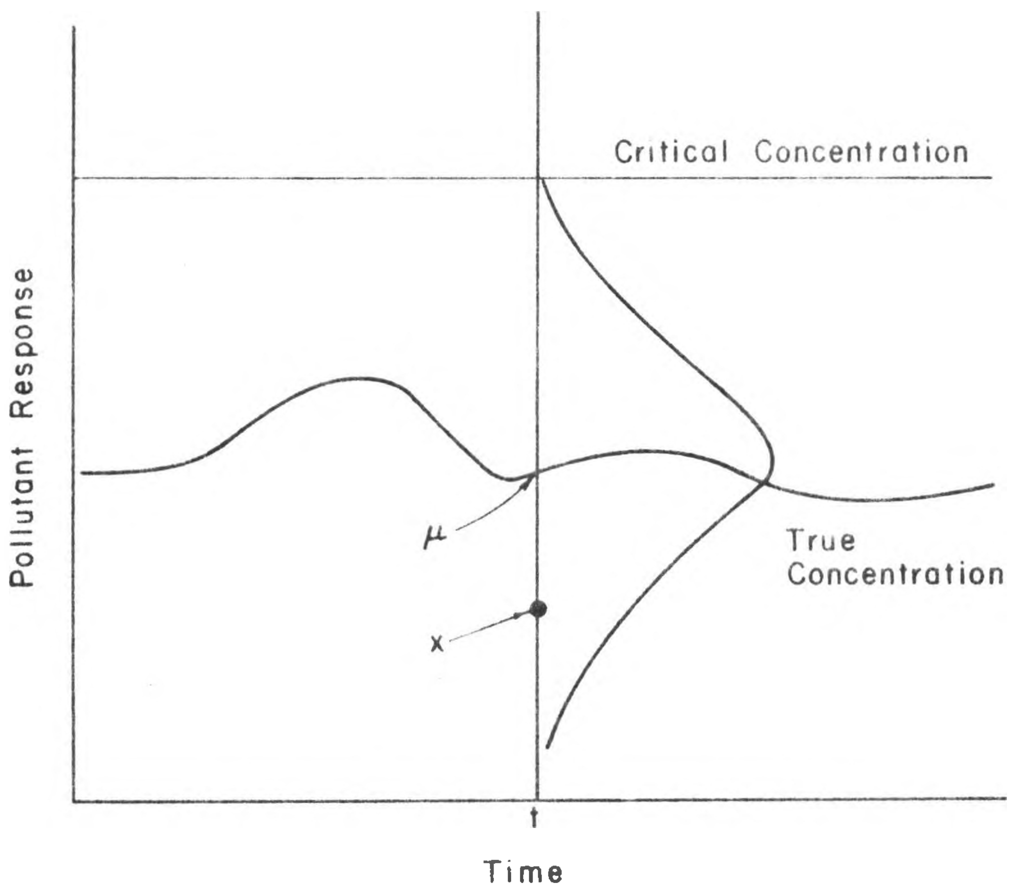

Figure 5. Plot of True Level of Pollutant Concentration vs. Time with an Associated Observation at Time t (Hunter, 1977).

considering only one distribution as shown in Figure 6. Failure concentration was set at $\mu + 3\sigma$ which gives a probability of occurrence of approximately 0.0013. This indicates that there is only a 0.13 percent chance that a concentration this high actually came from the population being considered. Therefore, there is justification in the assumption that such a high concentration did not come from the population generated when the process is working properly and could reasonably be considered as an indication of failure in that process.

The use of the $\mu + 3\sigma$ is a concept of failure which has been used in the design of effluent monitoring systems (Hunter, 1977). This concept of failure will be used in this study, even though it is recognized that other definitions may be more applicable. At this time, information available for justifying any definition of failure is limited.

The other critical value, detection concentration, was determined in the same way as failure concentration. However, in this case the restrictions were relaxed somewhat and a value of $\mu + 2\sigma$ was chosen. This gives a probability of occurrence of approximately 0.0227. If a concentration this extreme were detected, there would still be strong justification for assuming that it came from another distribution (in this case an indication of failure).

As previously stated the detection and failure concentrations used in this study were based on the distribution of septic tank effluent (input concentration into the leach field). From a study conducted by the SSWMP (1978) in which 108 total observations were taken from 7 septic tanks, the mean and standard deviation of ammonium-N concentrations were reported as 30 mg/ℓ and 15 mg/ℓ, respectively. These values result in a

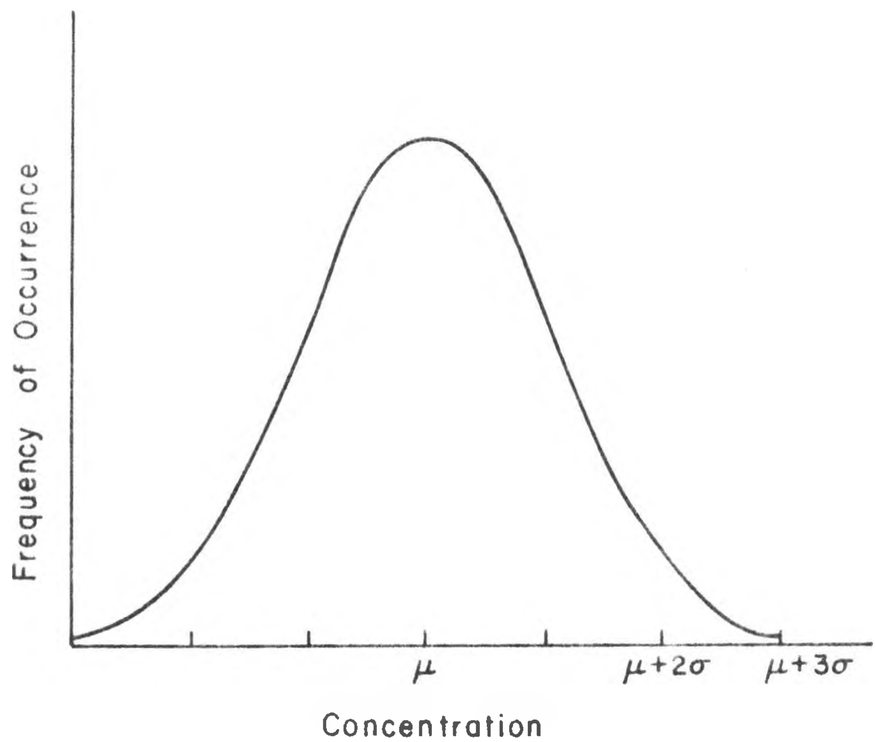

Figure 6. Distribution of Possible Observations from a Pollutant Process from which Critical Values can be Determined.

failure concentration of 75 mg/ℓ nitrate-N and a detection concentration of 60 mg/ℓ nitrate-N.

Random Components

The simulation of input to the mass transport model consisted of several random components. This first random component considered was the time variation of septic tank effluent concentration. No data were available on the distribution of effluent concentrations with time. Therefore, the nitrate-N concentration inputs to the model were determined as random deviates from a truncated normal distribution (0 mg/ℓ was taken as a lower limit). The mean and standard deviation used were 30 mg/ℓ and 15 mg/ℓ, respectively, given previously as the mean and standard deviation of ammonium-N concentration. The assumption that complete nitrification of the ammonium occurs within a few centimeters of the crusted layer, as discussed previously, was followed. A random deviate was taken from the parent distribution at three-day intervals so that the resulting concentration distribution with time was a step function as shown in Figure 7.

The other random components of each simulation were temporary system overloads and one permanent system failure which were superimposed on the input concentration. A more complete discussion of system failure is given later. However, for the purposes of the present discussion a temporary system overload is defined as a short term increase in input concentration beyond an acceptable level followed by a return to normal operating conditions. A system failure is an increase in concentration beyond an acceptable level which does not return to normal operating conditions. The concentration for both the temporary overload

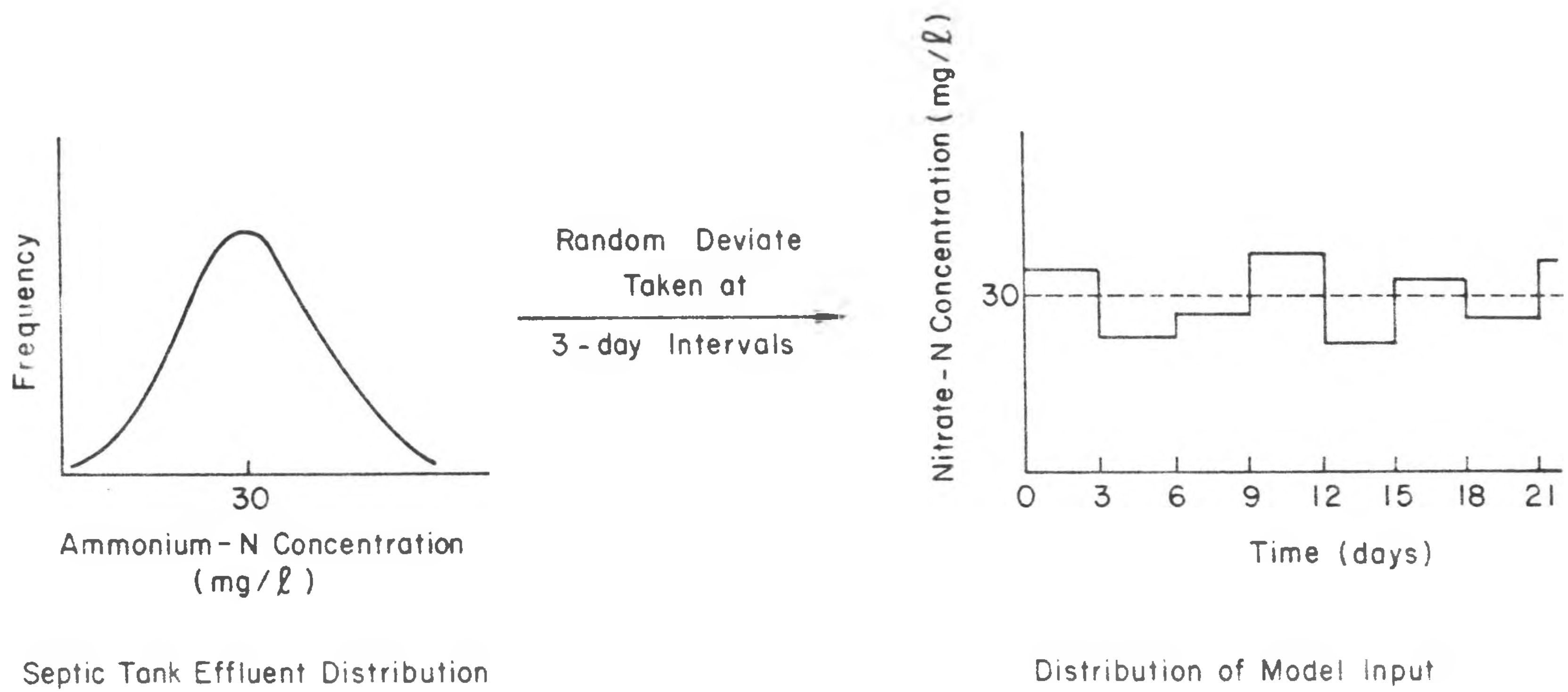

Figure 7. Simulation of Model Input Nitrate Distribution with Time.

and system failure was taken as the failure concentration so that the only difference is the permanence of the system failure.

A random number, from 1 to 50, of temporary system overloads were superimposed on each simulation of input concentrations. The beginning of each temporary overload was determined randomly on the interval (0, 365) days. The duration of each overlad was also random on the interval (0,3) days. Each simulation was operated over a one-year period during which time a system failure was superimposed on the process at a random time between 0 and 365 days. Once the system failure occurred operation continued under those conditions for the duration of that simulation run.

The beginning of each temporary overload, the duration of each overload, and the beginning of the system failure were all determined randomly on continuous intervals. Therefore, the final distribution of input concentration with time used in each simulation no longer varied on a three-day basis. Instead, the input consisted of a background concentration varying on a three-day basis superimposed with system overloads at random times.

Sensitivity analysis was conducted on all the random components of the model. A complete description of the changes considered is presented along with the results of those changes in Chapter 4.

Sampling Plans

The concentrations of each input simulation were run through the mass transport model resulting in a simulation of the concentration variability with time at the sampling point. A sampling plan was then superimposed on this output in order to determine the effectiveness of that plan in detecting system failure. Three different sampling plans were evaluated.

Plan I -- Samples were taken at equally spaced intervals at frequencies of 1, 3, 5, 10, 15, and 20 samples per year. If a concentration above the detection limit was found, system failure was assumed and sampling was terminated.

Plan II -- Primary samples were taken at the frequencies given in Plan I. However, if a concentration above the detection limit was found, the primary sample was followed by one secondary sample one week later. If both the primary and secondary samples were above the detection limit, system failure was assumed and sampling was terminated.

Plan III -- Primary samples were taken at the frequencies given in Plan I. However, if a concentration above the detection limit was found, the primary sample was followed by two secondary samples 3 days and 6 days later. If all three samples were above the detection limit, system failure was assumed and sampling was terminated.

Sampling frequencies were approximately equally spaced over the one-year time period considered in each simulation. For example, for a frequency of one sample per year the primary sample was taken at day 182 and for a frequency of 2 samples per year the primary samples were taken at day 121 and 242. For each sampling plan and sampling frequency, the percent failure detection, percent temporary overload detection, percent no detection, and average time from failure to detection were determined.

From the description of the various sampling plans considered, it is obvious that the primary objective of plans II and III is to avoid classifying a detection as a system failure when it is actually a temporary overload. It is important to note that the procedures used to develop monitoring strategies are necessarily very closely related to the monitoring objectives. For example, if the purpose of the inspection monitoring program were the detection of temporary failures, a different approach would be required. This type of problem would be similar to developing a monitoring program for the detection of pollution spills in streams as investigated by Vanderholm (1972). He determined that extremely high sampling frequencies were necessary to detect a failure of this temporary nature.

It is reasonable to assume that the same conclusion would be found in a ground water situation and the results presented in Chapter 4 imply such a conclusion. The cost involved in a monitoring program of this type would be prohibitive for a community management organization.

Simulation Example

Figure 8a shows a typical simulation of the input concentrations. This particular simulation resulted in two temporary system overloads. The first overload occurred at 1.7 days and had a duration of 1.8 days while the second overload occurred at 13.5 days and had a duration of 1.3 days. At the completion of a temporary overload, a random concentration is selected for the remainder of that day. Then, the three-day variation is resumed until the next overload. For the example shown, at day 17 the system failed and that concentration remained throughout the rest of the simulation period.

Figure 8b shows the distribution of the concentration at the sampling depth which resulted from the input distribution given in Figure 8a. The concentration was calculated from equation 4 at the beginning of each day. The detection concentration, i.e., the concentration above which failure was assumed to have occurred, is also indicated in the figure.

As can be readily seen from Figure 8b, any sample taken after day 23 would have correctly indicated system failure. However, a sample

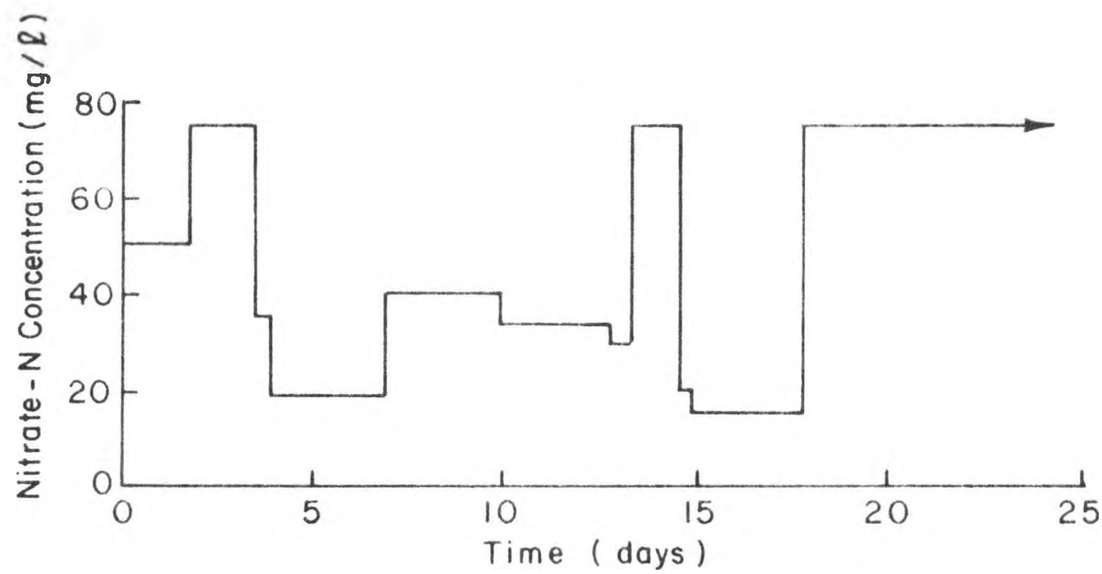

Figure 8a. Random Input Distribution with Time.

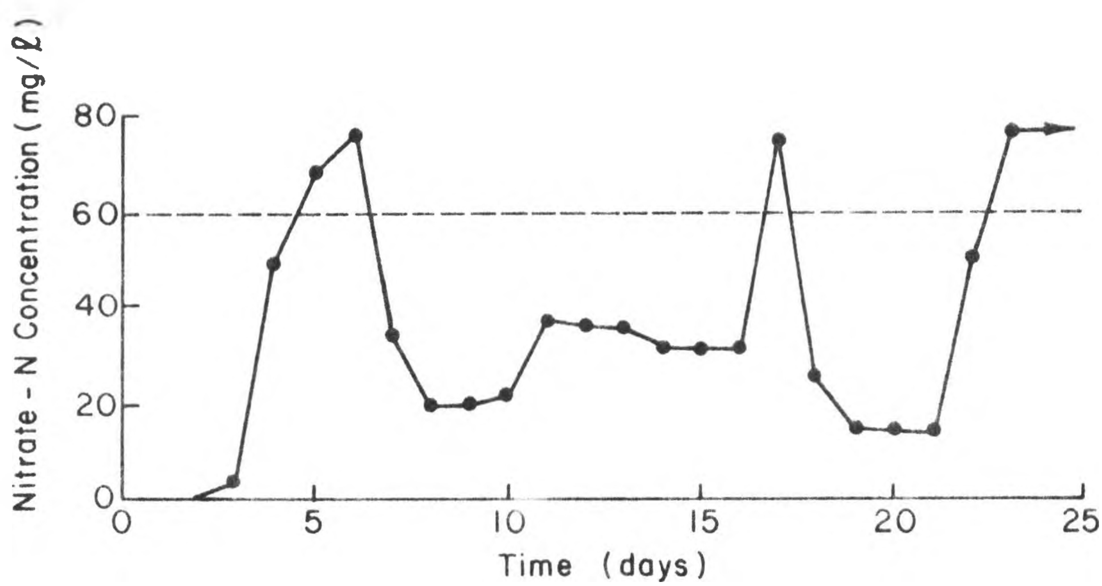

Figure 8b. Distribution with Time at the Sampling Depth as a Result of the Above Input Realization. (Detection Concentration Indicated by Dashed Line).

following plan I on day 5, 6, or 17 would also have indicated system failure. Sampling plans II and III would not have resulted in the false indications of system failure regardless of when the sample was taken.

Number of Simulation Runs

In designing a simulation study it is necessary to decide how many times to sample the system behavior, i.e., how many times to run the simulation. For this study the number of simulation runs were chosen to give the estimated percent failure detection to an accuracy of $\pm$ 5 percent at a 90 percent confidence level. The equation used for this determination was (Benjamin and Cornell, 1970):

$$n \geq \frac{Z^2_{\alpha/2}\ (1 - \hat{p})}{\gamma^2\ \hat{p}} \tag{5}$$

where n is the number of simulation runs necessary to insure that the $(1 - \alpha)$ 100 percent confidence limits are within γ100 percent of the true value of p, the percent failure detection. For the above equation $Z_{\alpha/2}$ is that value which a standardized normal random variable exceeds with a probability of $\alpha/2$ and for this study $\hat{p}$ is the estimated value of the percent failure detection.

A sequential procedure was used where an estimate of p was determined from the first set of simulation runs. From this first estimate a value of n was calculated and additional runs were made in order to satisfy this value of n. A new estimate of p was determined and equation 5 was recalculated. Additional runs were made if necessary. This process was repeated until enough runs had been made to satisfy the equation given the current $\hat{p}$.

The use of equation 5 requires the assumption of normality. The distribution being sampled is actually binominal since for each simulation, system failure was either detected or it was not. However, for large enough values of np the distribution is approximately normal. Therefore, the criterion is satisfied.

A complete listing of the computer program combining input simulation and mass transport is given in the appendix. A flow chart is shown in Figure 9 which briefly describes the operation of the model and the superposition of sampling plans on the model output.

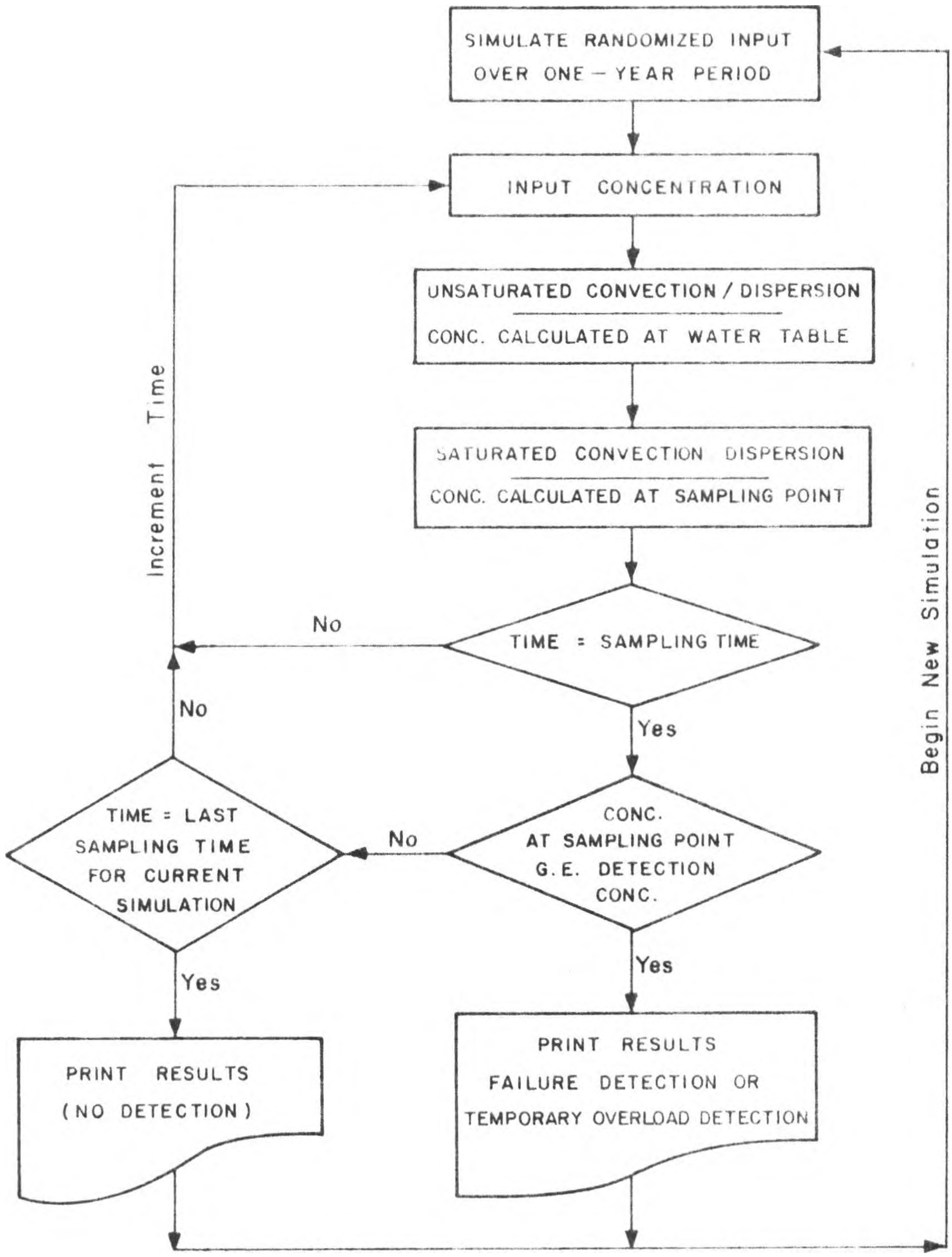

Figure 9. Flow Chart Describing Operation of the Model.

CHAPTER 4

INSPECTION MONITORING

As indicated in Chapter 3 the results obtained from the model were used to evaluate the effectiveness of various sampling strategies in detecting failure of individual on-site systems. This type of monitoring has been referred to as inspection monitoring to distinguish it from trend monitoring which will be discussed in the following chapter. The results of the model are presented and discussed in this chapter along with ramifications of assumptions made.

Presentation of Model Results

For each sampling plan considered four quantities were determined: the percent of failure detection, percent temporary overload detection, percent no detection, and average time from failure to detection. The percent failure detection serves as a crude estimate of the effectiveness of a sampling plan. However, it should be noted that given a continuous system failure as defined in this study, any sampling plan will eventually detect either a system failure or a temporary system overload. Each simulation was run over a one-year period; therefore, the percent failure detection can provide an evaluation of the sampling plans only over that limited time period.

Table 1 through Table 3 indicate the results of various sampling plans. A minimum confidence level of 90 percent was used as a criterion for the number of simulation runs. However, in many cases more simulation runs were made than necessary so that this level was exceeded. The actual confidence level attained is reported.

Primary Samples (per year)	Number of Simulation Runs	*Confidence Levels (%)	Percent Failure Detection	Percent Temporary Overload Detection	Percent No Detection	Average Time From Failure to Detection (days)
1	270	90.0	49.5	6.1	44.4	86.4
3	252	91.5	69.0	14.6	16.4	51.5
5	326	94.0	65.4	19.3	15.3	34.5
10	269	90.1	55.8	39.4	4.8	18.9
15	270	90.0	48.1	51.5	0.4	14.3
20	275	90.6	42.9	56.0	1.1	11.9

Table 1. Results of Simulation for Plan I - Primary Samples at the Indicated Frequencies with No Secondary Samples

*Confidence level such that the confidence interval for failure detection is $\pm$ 5%.

Primary Samples (per year)	Number of Simulation Runs	*Confidence Levels (%)	Percent Failure Detection	Percent Temporary Overload Detection	Percent No Detection	Average Time From Failure to Detection (days)
1	275	90.3	48.0	1.1	50.9	100.4
3	205	90.1	75.0	4.0	21.0	54.5
5	200	91.2	79.5	5.0	15.5	36.9
10	150	90.0	83.3	4.7	12.0	26.2
15	178	91.1	80.9	10.7	8.4	20.8
20	160	90.5	82.7	12.0	5.3	18.5

Table 2. Results of Simulation for Plan II - Primary Samples at the Indicated Frequencies Followed by One Secondary Sample

*Confidence level such that the confidence interval for failure detection is ± 5%.

Primary Samples (per year)	Number of Simulation Runs	*Confidence Levels (%)	Percent Failure Detection	Percent Temporary Overload Detection	Percent No Detection	Average Time From Failure to Detection (days)
1	270	90.0	47.2	0.0	52.8	93.1
3	215	90.1	73.1	1.0	25.9	58.5
5	150	94.0	88.0	0.0	12.0	37.2
10	137	96.0	91.1	0.8	8.1	25.1
15	100	93.4	92.0	3.0	5.0	19.9
20	100	94.9	93.0	1.0	6.0	17.3

Table 3. Results of Simulation for Plan III - Primary Samples at the Indicated Frequencies Followed by Two Secondary Samples

*Confidence level such that the confidence interval for failure detection is ± 5%.

As shown in Tables 1, 2, and 3 and in Figure 10, sampling plan III is the most effective in detecting system failure. With this plan a sampling frequency of at least 7 samples per year is necessary to detect a system failure 90 percent of the time on the average. Figure 10 also indicates that an increase in sampling frequency beyond this point would not be very beneficial in terms of increased failure detection.

It is also noted that sampling plan I reaches a maximum percent failure detection at a frequency of 3 or 4 samples per year and higher frequencies actually reduce the effectiveness. The reason for this apparent anomaly is that higher frequencies tend to begin detecting an increasing number of temporary system overloads as indicated in Figure 11.

The percent temporary overload detection is perhaps a more descriptive quantity than the percent failure detection. The temporary overload detection provides a measure of the error associated with the sampling plan. This error is analagous to the Type I error from basic statistics, i.e., an acceptable process has been declared unacceptable. In establishing monitoring guidelines, it is desirable to keep the probability of this type of error as small as possible. It was, in fact, this rationale which led to considering sampling plans in which the primary samples were followed by secondary samples so that a suspected system failure could be confirmed.

Given the randomized inputs used in this model, it is obvious from Figure 11 that sampling plan I serves rather poorly with regard to minimizing the type of error described above. Sampling plan II provides a drastic improvement but still may not be effective enough if the higher sampling frequencies are required. Sampling plan III, however, results in very low temporary overload detection even at high sampling freqencies.

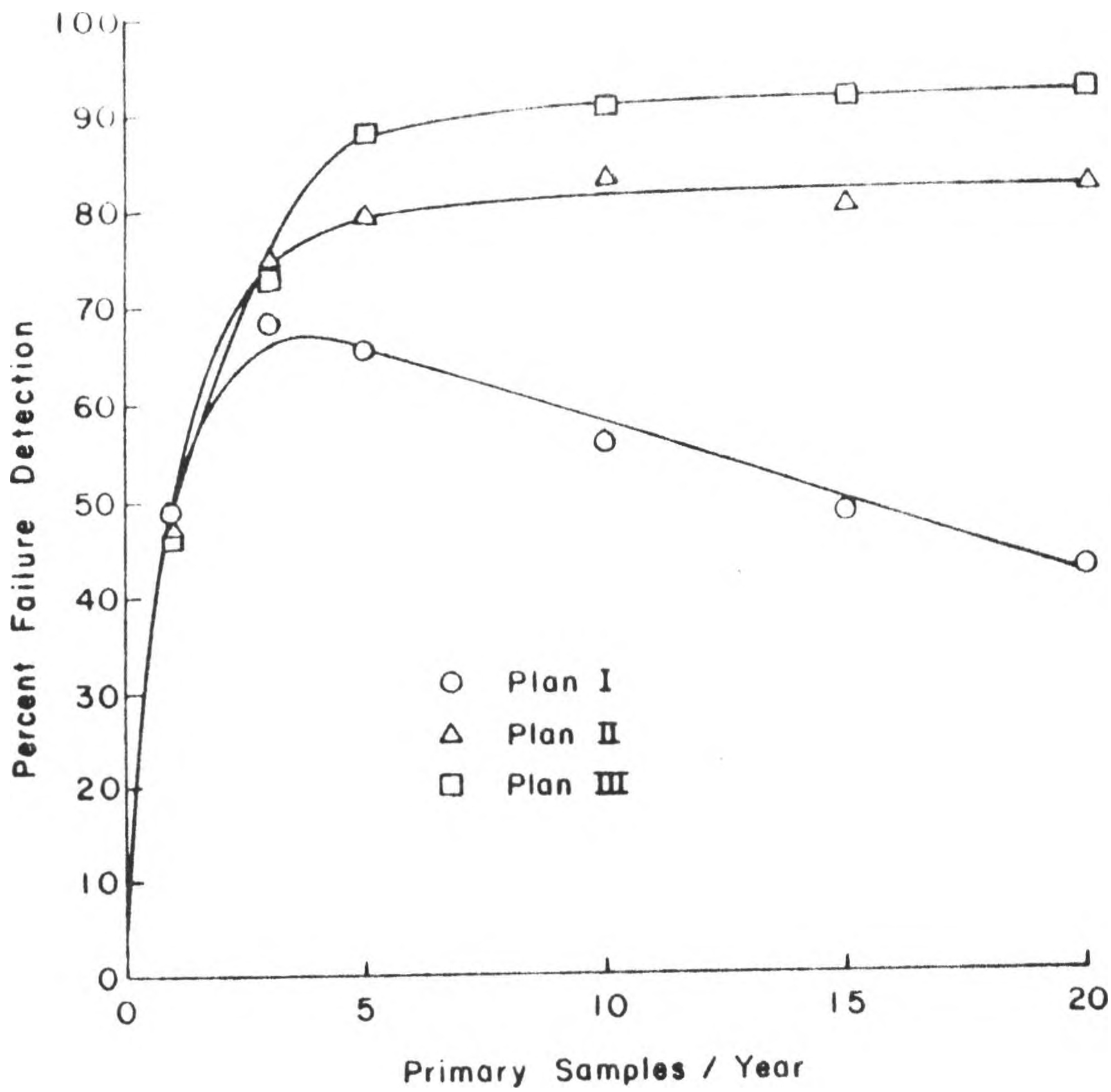

Figure 10. Comparison of the Effectiveness of Sampling Plans in Detecting System Failure.

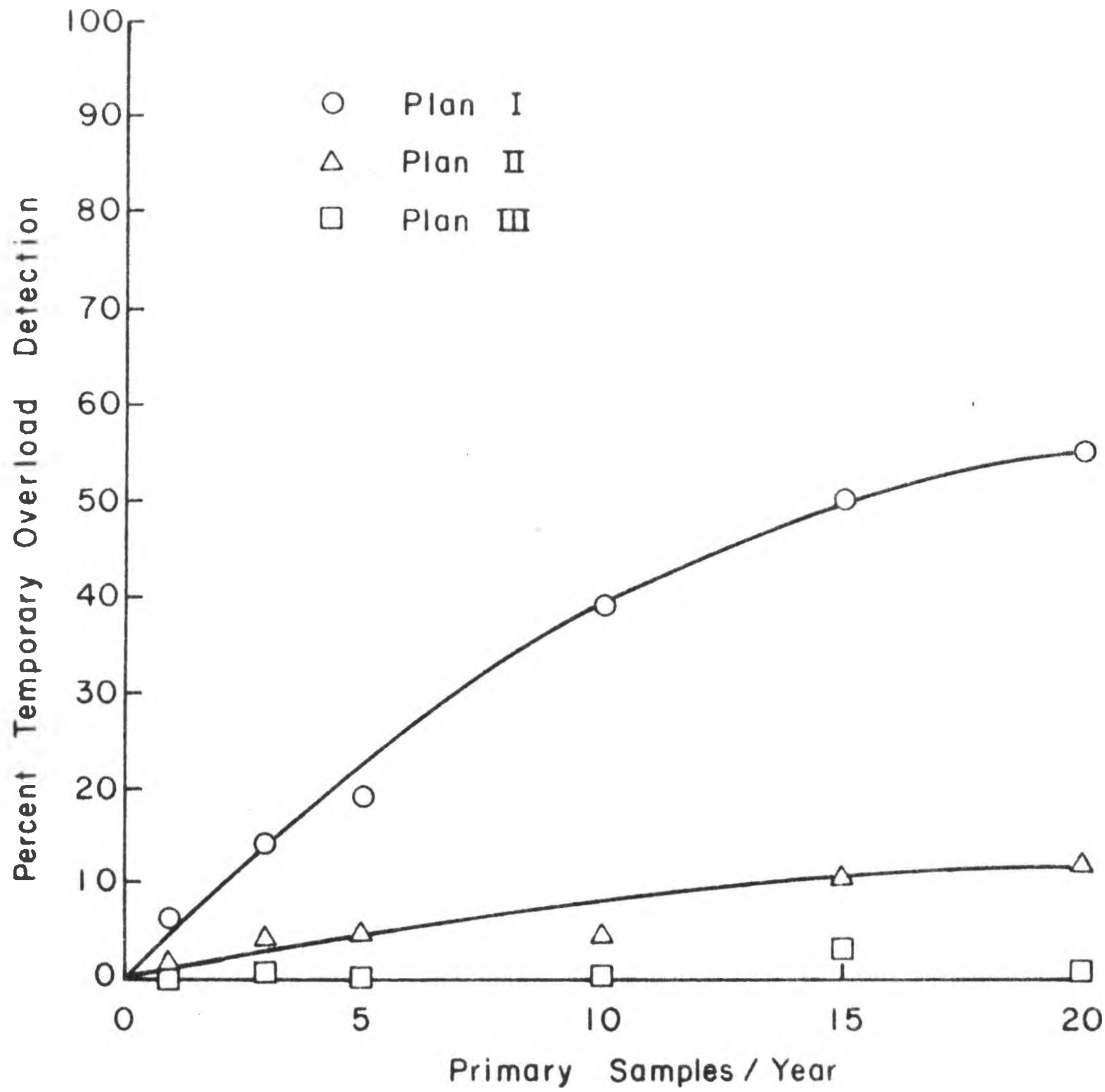

Figure 11. Comparison of the Effectiveness of Sampling Plans as Measured by Temporary Overload Detection.

Figure 12 represents the probability of no detection at all. As with the probability of failure detection, this is to some extent characteristic of the time frame selected for the model. Given a long enough period of time, any of the sampling plans will detect either a system failure or a temporary overload. For equal sampling frequencies Figure 12 shows that sampling plan I results in a samller percentage of no detection than either plans II or III. However, this is a result of plan I detecting more temporary overloads and, therefore, is not an indication of superiority of plan I.

Another factor considered in the evaluation of various sampling strategies was the average time after a system had failed until the sampling detected that failure. This quantity was only determined for the cases where a system failure was actually detected. Therefore, these results must be considered in conjunction with the results already presented in order to make valid interpretations. From Figure 13 it can be seen that sampling plans II and III lag behind plan I by about 6 or 7 days. This should be expected since sampling plans II and III required secondary samples to be taken and confirmation of system failure could only occur 6 or 7 days after a failure was initially detected.

The results shown in Figures 10 through 13 indicate that sampling plan I would be inadequate in detecting a system failure given the characteristics of the particular system that was modeled. Sampling plan II provides a significant improvement over plan I and may be suitable in some cases. However, if it is necessary to use higher frequencies in order to decrease the average time from failure to detection, it may be advantageous to use sampling plan III. From Figure 11 it can be seen that 20 samples per year results in an average

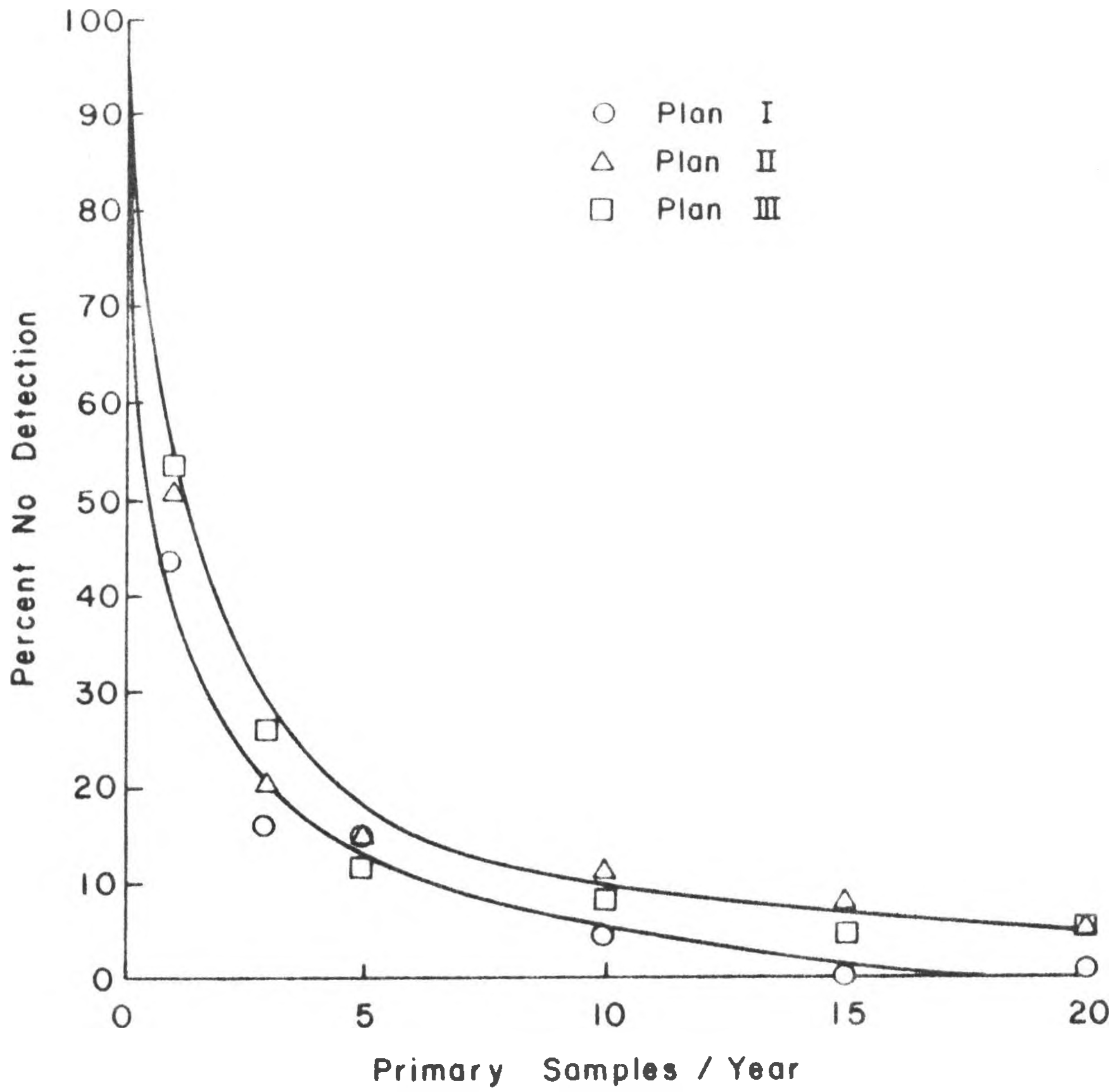

Figure 12. Comparison of the Effectiveness of Sampling Plans as Measured by No Detection. (The Differences Between Plan II and Plan III Were Not Deemed Significant Enough to Justify Two Curves).

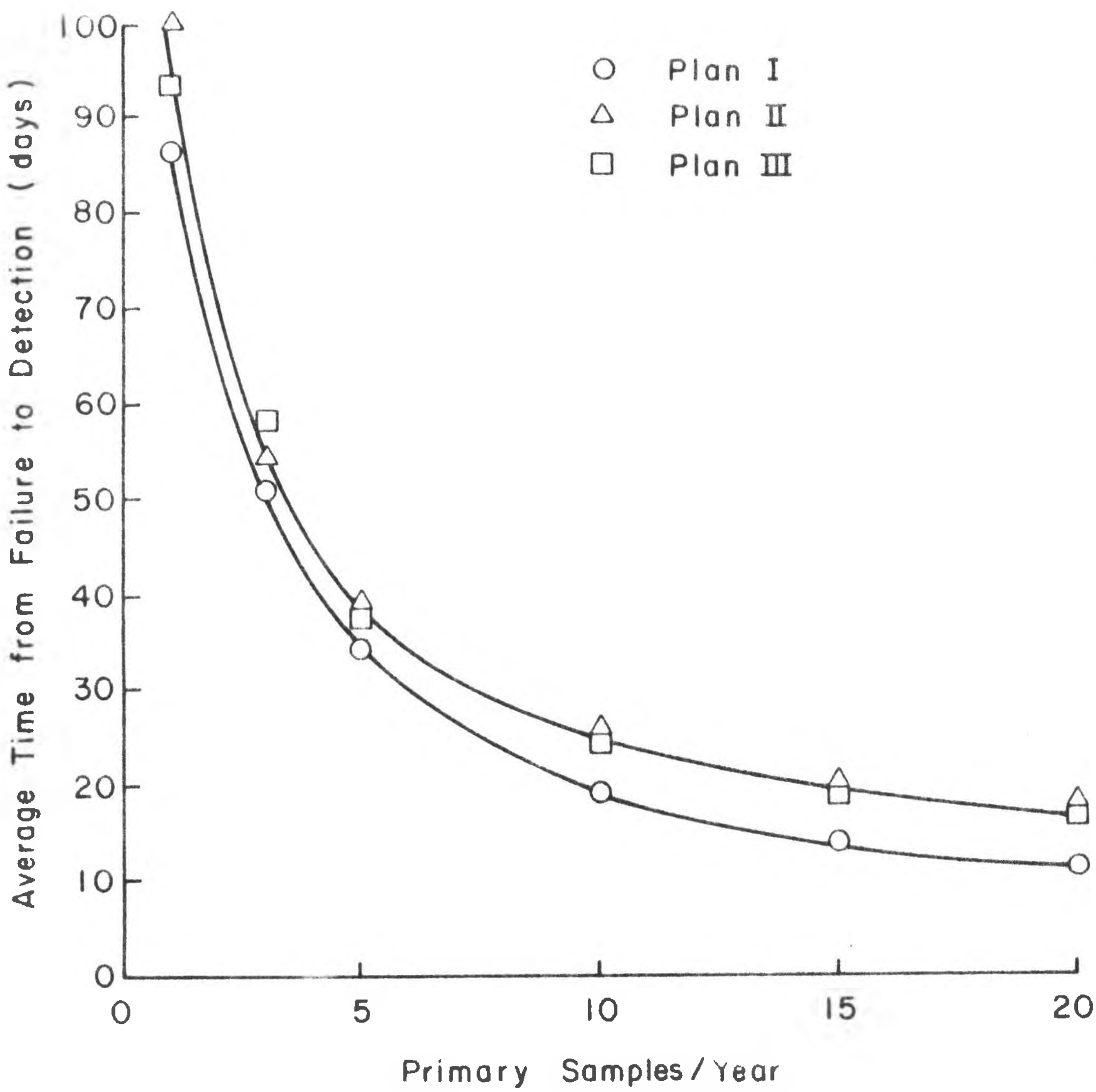

Figure 13. Comparison of Average Time from Failure to Detection for the Sampling Plans. (The Differences Between Plan II and Plan III Were Not Deemed Significant Enough to Justify Two Curves).

temporary overload detection of approximately 10 percent for plan II while for plan III the temporary overload detection is negligible.

Sampling plan III requires more samples, and consequently, would be more costly. The choice between plan II and plan III would be dependent on the value a management agency is willing to accept as the probability of making an error by classifying a detection as a failure when it is actually a temporary overload.

It was previously stated that the results presented here imply that sampling for temporary failures would require a frequency that would be too high to be feasible for a management organization. This conclusion is supported by the results shown in Figure 11. Failures of a limited duration would correspond to the temporary overloads used in this model. For detection of this type of failure there would be no need for secondary samples. Therefore, the results given for sampling plan I are indicative of the type of results which would be anticipated for a monitoring program set up to detect temporary failures. Figure 11 shows that unreasonably high sampling frequencies would be required in order to obtain an acceptable probability of detection.

Sensitivity Analysis

In order to establish the effectiveness of various sampling plans under different conditions, a sensitivity analysis was conducted. The effect of varying the parameters in the transport model were first studied. Changes in these parameters are equivalent to physical changes in the leach field. Two factors were considered--depth to ground water and soil type. The sensitivity analysis for these two factors was conducted on sampling plan III with a sampling frequency of 10 primary samples per year.

The effect of considering different input randomizations was also studied. The interval from which a random number of temporary system overloads was selected as well as the interval from which a random duration of each overload was selected were both changed. Results of these changes are given for all three sampling plans and for a frequency of 10 primary samples per year.

Transport Model Parameters

As indicated in Table 4, varying the depth to ground water had very little effect on the sampling plan. The ground water depth for the initial investigations (Figures 10 through 13) was set at 150 cm. Sampling plan III with 10 primary samples per year was tested at ground water depths of 90 cm and 250 cm. Recalling that the confidence interval for the percent failure detection is $\pm$ 5 percent, Table 4 indicates that there is no justification for assuming any difference in the effectiveness of this sampling plan at the ground water depths shown.

These results should be anticipated in view of the fact that the probability of detection is a function of the length of time a pollution event is in contact with the sampling point. For a given pollution event this time should not change as a result of an increase or decrease in the ground water depth, with the exception of the change that is brought about by dispersion. Apparently, dispersion was not significant enough to alter the effectiveness of the sampling plan.

These results are based on the assumption that the sampling point is located 20 cm below the water table regardless of the ground water depth. If the sampling point were located at a constant depth, it is reasonable to assume that a change in the water table could have a pronounced effect on the sampling plan depending on the extent of the

Ground Water Depth (cm)	Number of Simulation Runs	Confidence Levels (%)	Percent Failure Detection	Percent Temporary Overload Detection	Percent No Detection	Average Time From Failure to Detection (days)
150	137	96.0	91.1	0.8	8.1	25.1
90	149	98.5	93.3	0.0	6.7	25.1
250	120	94.1	90.8	1.7	7.5	26.5

Table 4. Results of Changing the Depth to Groundwater for Sampling Plan III with 10 Primary Samples per Year.

ground water flow and the dilution which would occur. However, the intent of this study was to design a monitoring program which would intercept a pollution event at maximum concentration before dilution had taken place. The particular ground water flow conditions at an individual site could then be taken into account to predict the ultimate impact on the aquifer. The physical problems associated with collecting samples at varying depths as the water table fluctuates is discussed later in this chapter.

The results shown in Table 4 indicate that there is very little difference in the average time from failure to detection as a consequence of changing the ground water depth. However, it should be noted that these results are based on the parameter values of a sandy soil. This would not be expected to be the case for a fine-textured soil with decreased seepage velocity.

Although these results indicate that the sampling plans are not affected by a change in ground water depths, this must be viewed in light of the fact that the sampling plans are based on detection of nitrates. It is imperative to maintain a suitable depth of unsaturated flow conditions for the removal of bacteria and viruses. Considering this point, it is suggested that one of the primary efforts of a management agency should be the monitoring of ground water depth.

The other change in the physical conditions of the leach field which was considered was the soil type. The soil type is defined in the transport model by the seepage velocity and dispersion coefficient. Therefore, a change in the type of soil assumed in the model was effected by changing the values of these parameters. The values selected were given in the preceding chapter. Initially, a sandy soil was assumed in the model. A silt loam soil was considered for the sensitivity analysis.

A unique problem was encountered in evaluating the effect of changing the soil type. As previously discussed, the results of the model are to some extent dependent on the time frame selected for the simulation runs. This became particularly evident in considering a fine textured soil, such as the silt loam soil used in the sensitivity analysis. Due to the much smaller seepage velocities which are representative of this type of soil, the lag time from the introduction of a slug of wastewater until that slug appeared at the sampling point was dramatically increased. Neglecting dispersion, a lag time of 79.4 days was calculated for the silt loam soil and 2.2 days for the sandy soil. Based on these calculations, a system failure which occurs any time after 287.6 days for the silt loam soil could not be detected within the one-year simulation period assumed in the model regardless of which sampling plan was used. However, a sampling program which extends beyond the one-year period would certainly detect this failure.

In order to circumvent this problem, the percent detections were calculated on a relative basis. This eliminates the bias toward the sandy soil, which is introduced by virtue of the time frame selected, and allows comparison of the sampling plans in the two types of soils. Given the lag times of 79.4 days and 2.2 days for the silt loam soil and the sandy soil, a maximum possible detection of 78.3 percent is calculated for the silt loam and 99.4 percent for the sandy soil. The relative percent failure detection was determined by dividing the percent failure detection (given in the model results) by the maximum possible detection.

Table 5 indicates that when considered on the basis of relative percent detection, the soil type makes very little difference in the effectiveness of the sampling plan. This follows from the same reasoning that was used in evaluating changes in the ground water depth. The increase in the probability of detecting a pollution event in a silt loam as opposed to a sandy soil should correspond to the increased time for a pollution slug to be in contact with the sampling point.

Considering this, one should anticipate that the probability of detecting a temporary system overload would be slightly greater in the silt loam soil as a result of slower velocities. From the results shown in Table 5 it can be seen that sampling plan III is still very effective in eliminating error as indicated by the low value for percent temporary overload detection. Apparently, the increased time for a pollution event to be in contact with the sampling point was not long enough to cause a significant increase in the probability of detecting a temporary system overload for this sampling plan.

The method used to present the results on a more comparative basis is obviously still somewhat biased since it allows the silt loam soil to be evaluated on a reduced time scale. However, the fact that no temporary overloads were detected during any of the simulation runs attests to the effectiveness of the sampling plan even if the reduced time scale is considered. This seems to indicate that the percent temporary overload detection would still be very low even if the sampling plans were subject to the full one-year time period.

Due to the increased lag time for a pollution event to reach the sampling point in a fine-textured soil, the average time from failure to detection is increased rather dramatically as shown in Table 5. This

Soil Type	Number of Simulation Runs	Confidence Levels (%)	Percent Failure Detection	Percent Temporary Overload Detection	Percent No Detection	Average Time From Failure to Detection (days)
coarse sand	137	96.0	91.1 (91.6)*	0.8	8.1 (7.6)	25.1
silt loam	230	90.0	69.9 (91.9)	0.0	30.4 (8.1)	117.5

Table 5. Results of Changing Soil Type for Sampling Plan III with 10 Primary Samples per Year.

*Numbers in parentheses refer to relative percent detections based on a maximum failure detection of 99.4% and 75.7% for coarse sand and silt loam, respectively. The values for percent no detection adjusted accordingly.

increased time is unavoidable. However, it should be noted that this time refers to the period from the occurrence of a pollution event at the input to the detection of that event at some point in the soil. This implies that the failure occurs before the wastewater enters the leach field. If the failure actually occurs at some point in the leach field itself, this particular analysis does not apply.

The results obtained from changing the physical parameters of the system indicate that these changes have a minimal effect on the efficiency of sampling plan III. No attempt has been made to extrapolate these results to other sampling plans and it is anticipated that plan I may be affected more by such changes. Sampling plan I does not require secondary samples to confirm system failure. Therefore, it would be expected that the increased contact time of a pollution event with the sampling point would have a much more pronounced effect on increasing the probability of detecting temporary overloads for plan I than for the other plans. Consequently, sampling plan I, which served poorly for the sandy soil, would be even less efficient in a fine-textured soil.

Input Randomizations

In the evaluation of sampling strategies, the effect of the input to the leach field was shown to be more important than the physical characteristics of the leach field. This factor was evaluated by considering different randomizations of the input to the model.

The first change, shown in Table 6, was an increase in the duration of a temporary system overload. Initially, the model was run such that the duration of each temporary overload was selected randomly from the interval (0,3) days. This interval was increased to (3,7) days for the sensitivity analysis. A duration selected randomly from the interval

Sampling Plan	Interval From Which Temporary Overload Duration in Selected (Days)	Number of Simulation Runs	Confidence Levels (%)	Percent Failure Detection	Percent Temporary Overload Detection	Percent No Detection	Average Time From Failure to Detection (days)
I	(0,3)	269	90.1	55.8	39.4	4.8	18.9
	(3,7)	235	90.1	31.7	66.0	2.3	19.8
II	(0,3)	150	90.0	83.3	4.7	12.0	26.2
	(3,7)	240	90.0	66.7	31.7	1.6	23.1
III	(0,3)	137	96.0	91.1	0.8	8.1	25.1
	(3,7)	238	90.1	67.8	26.7	5.5	25.1

Table 6. Results of Changing Interval From Which Temporary Overload Duration is Selected from (0,3) Days to (3,7) Days. Results are for Sampling Frequency of 10 Primary Samples per Year.

(3,7) was assumed to be an extreme condition. It was felt that temporary overloads of durations longer than this would be an indication of such poor operation that system failure could be assumed even though continuous operation under failed conditions did not exist.

Table 6 shows the effect of this change for all three sampling plans. Results indicate a dramatic increase in the percent temporary overload detection with a corresponding decrease in the percent failure detection. Sampling plan III is still the most effective of the three. However, even this sampling plan resulted in a temporary overload detection of 26.7 percent. Obviously, the increase in the number of temporary overloads detected is a result of the longer duration of these overloads. An increase in the time between the primary and secondary samples would have undoubtedly improved the effectiveness of sampling plans II and III.

The second change considered was with regard to the number of temporary system overloads. In the initial evaluation of sampling plans the number of temporary overloads for each simulation was selected randomly from the interval (1,50). The upper end of this interval was considered the extreme and a system which exceeded this number of temporary overloads would be operating so poorly that failure could be assumed even though the system would not be continuously failing. Therefore, for the purpose of sensitivity analysis, the number of temporary overloads was selected randomly from the reduced interval of (1,10).

The results are shown in Table 7. As anticipated, a significant improvement was found in sampling plan I. A slight improvement was shown for plan II and no improvement was indicated for plan III. However, reference to the percent temporary overload detection for

Sampling Plan	Interval From Which Number of Temporary Overloads is Selected	Number of Simulation Runs	Confidence Levels (%)	Percent Failure Detection	Percent Temporary Overload Detection	Percent No Detection	Average Time From Failure to Detection (days)
I	(1,50)	269	90.1	55.8	39.4	4.8	18.9
	(1,10)	180	90.0	78.9	14.4	6.7	19.2
II	(1,50)	150	90.0	83.3	4.7	12.0	26.2
	(1,10)	120	90.1	87.5	0.8	11.7	26.5
III	(1,50)	137	96.0	91.1	0.8	8.1	25.1
	(1,10)	112	92.5	90.2	0.0	9.8	25.1

Table 7. Results of Changing Interval From Which the Number of Temporary Overloads is Selected from (1,50) to (1,10). Results Are For Sampling Frequency of 10 Primary Samples per Year.

sampling plan III reveals that this plan is already operating at peak effectiveness. Therefore, a decrease in the number of temporary system overloads simply has no effect on this particular plan.

The final change considered in the sensitivity analysis was a combination of the above two. The interval from which the number of temporary overloads was randomly selected was decreased from (1,50) to (1,10) and the interval from which the duration of each overload was selected was changed from (0,3) days to (3,7) days. The results, shown in Table 8, indicated that these two changes tend to counteract each other and very little difference in the effectiveness of the sampling plans shows up as a consequence of these alterations.

Sensitivity Analysis Discussion

The results of the sensitivity analysis suggest that in developing monitoring strategies the physical parameters of the system are relatively unimportant. The same information may be obtained from a particular sampling plan regardless of the type of soil or the depth to the ground water (assuming that the sample is taken at the location specified in the model). However, the type of failure and the variability associated with that failure are of extreme importance.

Some assumptions were made which were not evaluated in the sensitivity analysis. The soil system was assumed to be homogenous and the effluent was assumed to be uniformly distributed over the leach field. The effect of violation of these assumptions on the sampling program is difficult to ascertain. On the basis of the results obtained in the sensitivity analysis of two different soil types, it would appear that nonhomogeneity would have a very limited effect on the final outcome of a sampling program. This also assumes that the nonhomogeneity does not

Sampling Plan	Interval From Which Number and Duration of Temporary Overloads is Selected	Number of Simulation Runs	Confidence Levels (%)	Percent Failure Detection	Percent Temporary Overload Detection	Percent No Detection	Average Time From Failure to Detection (days)
I	(1,50) & (0,3)	269	90.1	55.8	39.4	4.8	18.9
	(1,10) & (3,7)	255	90.2	63.3	31.1	5.6	20.7
II	(1,50) & (0,3)	150	90.0	83.3	4.7	12.0	26.2
	(1,10) & (3,7)	110	90.7	89.1	3.6	7.3	27.9
III	(1,50) & (0,3)	137	96.0	91.1	0.8	8.1	25.1
	(1,10) & (3,7)	140	98.4	93.6	1.4	5.0	24.8

Table 8. Results of Changing the Intervals From Which Both the Number and Duration of Temporary Overloads Are Selected. Results Are For Sampling Frequency of 10 Primary Samples per Year.

severely alter the flow path to the extent that the bulk of the contamination bypasses the sampling point.

The assumption of uniform distribution of the effluent over the leach field is critical and violation of this assumption could have a very definite effect on the results obtained from the sampling program. The model is set up to describe flow as if it were in a column and the results are extrapolated to field conditions. Obviously, deviation from uniform effluent distribution could result in nonrepresentative samples. However, the importance of uniform distribution extends beyond the application of a sampling program. It is essential to the prevention of localized overloading and short-circuiting of the effluent.

The failure considered in this discussion has been defined as continuous operation of a system under unsatisfactory conditions of high nitrogen concentrations. Several factors in this definition are critical and, if altered, would possibly indicate the need for a change in the monitoring strategy.

As previously discussed, the continuous nature of the failure necessitated the development of sampling plans which would distinguish between such a failure and a temporary overload. Monitoring strategies to detect system failures of a temporary nature would require sampling frequencies in excess of that which could reasonably be taken by a management agency.

It was proposed in Chapter 2 that the sampling program would be developed on the basis of detection of nitrates. It is again emphasized here that this is not intended to imply that only nitrates should be monitored. Sampling programs should include analysis for bacteria and in some cases other variables such as viruses and phosphorus. The specific variables which need to be monitored will be dictated by local conditions and considerations.

Failure has been specified as an unacceptably high nitrogen concentration. The possible causes of this failure have not been delineated. However, the location of this failure within the model has been restricted to some point before entry of the wastewater into the soil. In the case of excessively high nitrogen concentrations this seems to be a logical assumption since there is no reason for an increase in the nitrogen concentration in the soil.

This type of analysis would appear to justify an argument for sampling at the effluent point of the septic tank rather than beneath the leach field. However, recalling that other variables will also be monitored and consideration of the leach field as a part of the treatment system invalidates this approach. Also, it should be realized that with regard to sampling for nitrates, the worst possible condition (i.e., complete nitrification with no denitrification) was assumed in the model. This might not be the case at a particular location and sampling beneath the leach field will provide site specific information as to the actual concentrations which are being introduced into the aquifer. Hence, the effectiveness of the entire treatment system is being monitored.

Application of Results

The results given in this chapter provide a comparison of three different sampling plans and several sampling frequencies. These results are meaningful for the type of failure specified and for the various conditions set forth in the model. They can be used by a management agency to determine the effectiveness of various sampling plans and the increase in sampling precision which can be expected with an increase in frequency of sampling. Information of this type can provide a basis on which management can determine the allocation of available resources.

It is important to realize that the results obtained from an inspection monitoring program alone will not necessarily provide enough information for management decisions. However, the information gained from such a monitoring effort can be used in conjunction with other data, such as septic tank density and ground water flow rate for a particular area, to determine action which should be taken by a management organization.

In particular, it is suggested that monitoring results be combined with mass balance calculations to determine the effect the pollution load from a system will have on the aquifer. Assuming steady state conditions the concentration in the ground water can be expressed as

$$C_g = \frac{Q_i C_i + Q_e C_e}{Q_i + Q_e} \qquad (6)$$

where C_g = concentration of the ground water after mixing with the effluent

C_i = concentration of the ground water before mixing

Q_i = rate of flow of ground water

C_e = concentration of the effluent determined at the water table (results from inspection monitoring)

Q_e = rate of flow of effluent into the ground water.

Mass balance calculations can be used to determine either a limit on the number of systems to be allowed in a given area or the number of systems which can be allowed before requiring conversion to innovative systems which are more efficient in nitrogen removal.

The results presented in this chapter indicate that in order to establish an efficient inspection monitoring program, it will be necessary to accurately quantify the input distribution of an individual system. With the input specified, sampling plans can be accurately evaluated. In the initial stages of a monitoring program it will be necessary to estimate this input and the probability of failure of individual systems and allocate samples accordingly. As the data base increases, the validity of assumptions made can be checked and the sampling plan modified if necessary. Through such an iterative procedure a very effective monitoring program can eventually be established.

Physical Aspects of Inspection Monitoring

Throughout the analysis of all the results in this study it is, of course, assumed that adequate sampling procedures are followed so that representative samples are obtained. This requires adequate design and installation of monitoring wells and correct sampling methods. These topics have been discussed in depth by Diefendorf and Ausburn (1977). Since the same methods apply to trend monitoring, a detailed discussion is postponed until the next chapter.

To meet the requirement of sampling just beneath the water table, either a cluster of wells installed at various depths or a ground water profile sampler as developed by Hansen and Harris (1974) can be used. This particular sampler consists of a well point filled with sand and divided into sections with petitions made of caulking. A sampling probe is located within each section and tubing extends from each probe to the ground surface. A complete description of the construction of a profile sampler is given in the above reference.

One advantage of using a cluster of wells is that they can be located at various points in the leach field to give more extensive areal coverage. The number of wells necessary at each site (or the number of probes necessary in a profile sampler) will depend on the extent of water table fluctuation at that site.

CHAPTER 5

TREND MONITORING

Adequate management of ground water quality requires more extensive results than those which can be obtained from an inspection monitoring program alone. A complete description of ground water quality will also require trend monitoring which will be necessary to indicate over an extended period of time the collective effect of all the systems within the management jurisdiction. Although trend monitoring has historically referred to that monitoring which measures the effects of pollution input to a water resource with respect to time, these effects are important over the entire aquifer. Therefore, for purposes of this discussion, both temporal and spatial considerations will be incorporated into trend monitoring.

The primary constraint in establishing a trend monitoring program is the lack of data. Ground water quality data which has been collected in the past has been for very specific ground water related problems. Routine data collection over a time period which is extensive enough to be very useful in developing monitoring strategies has been virtually nonexistent. Consequently, the discussion provided herein is limited to a theoretical discussion of the types of analyses which can be applied to the ground water trend monitoring problem. As more data (e.g., data covering many variables and stations over a long period of time) becomes available these procedures can be refined to better match the statistical characteristics inherent in that data.

Previous Studies

An extensive amount of work has been done in the development of general ground water monitoring guidelines. LeGrand (1968) has discussed ground water quality monitoring approaches based on a conceptional model. A series of reports have been published concerning various aspects of ground water monitoring (Todd et al., 1976; Everitt et al., 1976; Hampton, 1976; Crouch et al., 1976; Tinlin, 1976). These reports thoroughly discuss several topics of concern in ground water monitoring including economic analyses, data management, and sampling methods. However, a discussion of the statistics involved in developing a monitoring program, which is the primary objective of this study, has not been included.

The basic report in this series is Monitoring Groundwater Quality: Monitoring Methodology (Todd et al., 1976) which proposes fifteen steps toward developing a ground water monitoring strategy to be followed in chronological order. Since the procedure was designed to accommodate the monitoring of any type of pollution, it is necessarily very general. The purpose of the following is to list the steps of the procedure and briefly discuss them in relation to ground water monitoring strategies to support community management of on-site systems.

The steps included in the procedure are as follows:

Step 1 -- Select Area of Basin for Monitoring

Step 2 -- Identify Pollution Sources, Causes, and Methods of Waste Disposal

Step 3 -- Identify Potential Pollutants

Step 4 -- Define Ground Water Usage

Step 5 -- Define Hydrogeologic Situation

Step 6 -- Study Existing Ground Water Quality

Step 7 -- Evaluate Infiltration Potential for Wastes at the Land Surface

Step 8 -- Evaluate Mobility of Pollutants from the Land Surface to the Water Table

Step 9 -- Evaluate Attenuation of Pollutants in the Saturated Zone

Step 10 -- Prioritize Sources and Causes

Step 11 -- Evaluate Existing Monitoring Programs

Step 12 -- Establish Alternative Monitoring Approaches

Step 13 -- Select and Implement the Monitoring Program

Step 14 -- Review and Interpret Monitoring Results

Step 15 -- Summarize and Transmit Monitoring Information

Several of these steps will be applicable to both trend monitoring and inspection monitoring. Therefore, discussion of these steps will further serve to establish the relationship between these two types of monitoring.

The first three steps listed above identify the purpose of a particular monitoring program. For community management of on-site systems, this will be defined at the outset since these steps will coincide with the formulations of the objectives of a management agency. However, some considerations regarding these points should not be overlooked.

The area to be monitored will be specified as the jurisdiction of the management agency. However, it might also be necessary to divide the jurisdiction into smaller areas based on physiographic considerations. For example, it is conceivable that the management jurisdiction could cover several ground water basins. If this is the case, it would be desirable to consider each basin separately. If a single basin is large enough, it could be advantageous to divide it into smaller units for administrative convenience as well as statistical considerations to be discussed later. For management agencies with large areas under their jurisdiction it might also be necessary to prioritize areas for monitoring in order to operate within the budgetary limits that will most certainly exist.

Since the purpose of the agency will be management of on-site disposal systems, the primary pollution source and method of disposal have already been identified. However, in considering trend monitoring, it is important to realize that many other sources in the area may be contributing to the pollution load on the aquifer. Therefore, it will be of extreme importance to identify these sources. This points to the necessity of including both trend monitoring and inspection monitoring for effective overall management. Todd et al. (1976) discusses types of pollution which may be expected from municipal, agricultural, and other sources.

The potential pollutants from on-site systems have already been identified in considering inspection monitoring. Monitoring for bacteria and nitrates will be necessary for all situations. For trend monitoring, chlorides will be valuable as an indication of the extent of pollution. Site specific situations may warrant including other variables such as phosphorus or viruses. Also, if other pollution sources are suspected, it might be necessary to isolate a variable which is characteristic of on-site systems but not typically found in the other wastes.

Step 4 through Step 10 will provide an accurate description of the physical setting in which the monitoring program will operate. Given the budgetary constraints which will be imposed on the implementation of monitoring strategies, the ground water usage becomes a key factor in prioritizing monitoring needs. For example, if the major use of the ground water in a particular area is for drinking purposes, more intensive monitoring will be called for than if the major use is irrigation.

Defining the hydrogeologic situation will help to quantify the pollution potential for a particular area, as will the evaluation of infiltration potential; mobility, and attenuation of pollutants. These steps will all serve to establish priorities regarding which areas should be more heavily monitored.

Other methods for evaluating the potential for pollution can be incorporated as well. LeGrand (1964) developed an empirical method for determining pollution potential at a given site using a point-count system. Five factors influencing pollution are included in the method: depth to water table, water table gradient, distance from source, and two factors based on soil type--sorption and permeability. A value is associated with each factor and the points are summed. The total points are associated with the possibility for pollution. The method is based on results obtained from field studies. Although the approach is crude, it provides a valuable means of establishing monitoring priorities particularly in areas where data are limited. Defining the hydrogeologic situation only serves as a tool in developing monitoring guidelines and should not become an end in itself and thus an obstacle in the attainment of the overall objectives.

Most of the procedures listed in Steps 4 through 10 will have been performed for various sites throughout the basin as a result of inspection monitoring efforts. However, for trend monitoring general overall aquifer characteristics will be important as well as site specific observations.

The remaining steps relate to the actual implementation of a monitoring program. Although this study primarily focuses on the technical aspects of monitoring, the implementation of a monitoring program and the final use of the information obtained to maintain or improve water quality cannot be overemphasized. This will involve coordination with other monitoring programs which may exist for a particular area so that overlapping functions are avoided. Review and interpretation of the results by competent personnel will be required so that accurate decisions can be made regarding quality trends, new pollution problems, areas of improvement, and effectiveness of pollution control measures.

Although the procedure outlined by Todd et al. (1976) provides a very straight forward and logical approach to developing any ground water monitoring program, one essential element has been omitted. The statistics involved in the design have not been included. In order to develop monitoring programs so that the results can be obtained with a specified degree of confidence, it is necessary to incorporate statistics in the design.

Numerous studies exist on the application of statistics to the design of surface water monitoring networks. However, to date, only one source (Nightingale and Bianchi, 1979) has been located which considers the use of statistics in developing ground water monitoring programs. This investigation assumed complete independence of all observations and included no discussion of the ramifications of this assumption. It is the purpose of the remaining sections of this chapter to present a more complete evaluation of the statistics involved in developing ground water monitoring strategies.

Determination of Number of Samples Required

The primary statistical requirement of designing a ground water monitoring system is the establishment of the number of samples required in the context of both space and time. It has been suggested in previous reports (Ward et al., 1976; Sherwani and Moreau, 1975; Nightingale and Bianchi, 1979) that the number of samples required should be based on a specified confidence interval about the mean of the variable under consideration. This approach requires the selection of an acceptable precision with which the sample mean estimates the true population mean as well as the specification of a confidence level.

A $(1 - \alpha) \times 100$ percent confidence interval about the sample mean, $\bar{x}$, is given by

$$\bar{x} - Z_{\alpha/2}\,[\mathrm{Var}(\bar{x})]^{1/2} \leq \mu \leq \bar{x} + Z_{\alpha/2}\,[\mathrm{Var}(\bar{x})]^{1/2} \qquad (7)$$

where $\bar{x}$ = sample mean

μ = true population mean

$\mathrm{Var}(\bar{x})$ = variance of the sample mean

$Z_{\alpha/2}$ = the standard normal deviate corresponding to a probability of $\alpha/2$.

Equation 7 expresses that there is a $(1 - \alpha)$ probability that the true population mean will lie within the boundaries given. Use of this equation requires the assumption that $\bar{x}$ is normally distributed. This assumption is generally accepted as a result of the Central Limit Theorem which states that for any population with a finite standard deviation the distribution of the sample mean tends to normality as the number of observations, n, increases. The size of n required depends on the underlying distribution of the random variable. The further the departure from normality, the larger the value of n required.

There is no completely safe general rule as to how large n must be for the use of the normal approximation in computing confidence intervals. However, a crude rule is given by Cochran (1977) for populations in which the principle deviation from normality is a definite positive skewness. (Sherwani and Moreau (1975) note that positive skewness is typical of the distribution of many water quality variables). The number of observations should be selected such that

$$n > 25G_1^{\,2} \tag{8}$$

where G_1 = Fisher's measure of skewness.

$$G_1 = \frac{1}{Ns^3} \sum_{i=1}^{N} (x_i - \bar{x})^3$$

where N = the number of observations from the original distribution

s = the sample standard deviation

x_i = the i^{th} observation.

This rule is designed such that a 95% confidence probability statement will be wrong no more than 6% of the time.

Use of equation 7 obviously requires that the variance of the sample mean must be known. The expression for $Var(\bar{x})$ will vary depending on the correlation structure of the random variable in question. However, the number of observations will always appear in the expression. Hence, it will be possible to determine the number of samples required to meet a specified confidence limit given that the correlation structure is known.

Independent Observations

If the observations taken for the variables under consideration can be assumed independent in both space and time (i.e., no spatial or serial correlation exists), then the expression for the variance of the sample mean is

$$\mathrm{Var}(\bar{x}) = \frac{\sigma^2}{n} \tag{9}$$

where σ^2 = population variance

n = number of samples.

In general, the population variance, σ^2, will not be known but will have to be estimated from the data by the equation

$$s^2 = \frac{\sum_{i=1}^{n} (x_i - \bar{x})^2}{n - 1} \tag{10}$$

where s^2 = sample variance.

Replacing $\mathrm{Var}(\bar{x})$ with $\frac{\sigma^2}{n}$ in equation 7 and rearranging terms yields the following expression for determining the required number of observations:

$$n \geq \left[\frac{Z_{\alpha/2}\ \sigma}{|\mu - \bar{x}|} \right]^2 . \tag{11}$$

Substitution of s in place of σ in equation 11 requires the use of $t_{\alpha/2}$ instead of $Z_{\alpha/2}$, where $t_{\alpha/2}$ is the value of the student's t distribution corresponding to a probability of $\alpha/2$. Also, the difference between the true population mean and the sample mean, $|\mu - \bar{x}|$, is sometimes referred to as the precision, P. These substitutions result in

$$n \geq \left[\frac{t_{\alpha/2}\ s}{P} \right]^2 \tag{12}$$

from which it can be seen that three quantities are required to determine n. An acceptable confidence level, $1 - \alpha$, must be specified as well as the precision, P, which is deemed necessary. These two quantities can be determined as the values which the management agency is willing to accept. The third quantity, s^2, is an estimate of the population variance and must be determined from data.

If it can be assumed that the variance has not changed with time, the simplest approach for determining s^2 is to use historic data. If this data is not available, it may be possible to obtain at least a rough estimate of the population variance from previous sampling of a similar population or from a pilot survey.

Assuming no data are available, the most reliable estimate may be obtained by taking a sample in two steps. The first sample is a simple random sample of size n_1 from which an estimate of σ^2 is obtained. The required total sample size, n, is thus determined. Using this approach, Cochran (1977) shows how to compute n so that the final estimate of $\bar{x}$ will have a preassigned coefficient of variation or a preassigned variance.

If the coefficient of variation, CV, is specified and ${s_1}^2$ is the estimated variance from the first sample, additional observations should be taken so that the total sample size is

$$n = \frac{{s_1}^2}{(CV)^2 \bar{x}_1^{\,2}} \left(1 + 8(CV)^2 + \frac{s_1}{n_1 \bar{x}_1^{\,2}} + \frac{2}{n_1} \right). \tag{13}$$

where $\bar{x}_1$ is the mean obtained from the first sample and all other quantities are as previously defined. This procedure does result in an estimate for the population mean, $\bar{x}_b$, which is slightly biased. Consequently, the final estimate of the population mean should be taken as

$$\bar{x} = \bar{x}_b \, [1 - 2(CV)^2]. \tag{14}$$

These results assume that the population is normally distributed.

In order to determine the required number of samples to obtain a specified variance of $\bar{x}$ (which, in turn can be related to the confidence interval by equation 7) additional observations should be taken to make the total sample size

$$n = \frac{s_1}{\mathrm{Var}(\bar{x})}\left(1 + \frac{2}{n_1}\right). \tag{15}$$

If the population variance, σ^2, were known exactly, the required sample size would be $\frac{\sigma^2}{\mathrm{Var}(\bar{x})}$. The factor $(1 + \frac{2}{n_1})$ is necessary as a consequence of having to estimate σ^2.

Spatially Correlated Observations

For the preceding discussion the assumption of independent observations has been made. If samples are correlated, part of the information obtained from one observation is contained in other observations. As a result the sample size has to be increased in order to achieve the same information that could be obtained from uncorrelated observations. For sampling wells located within reasonable proximity of one another, one would not expect the independence assumption to be valid. Rather some spatial (interstation) correlation would be anticipated. The extent of interstation correlation which exists would be dependent primarily on the flow that occurs between wells.

For the case where there is spatial correlation, equation 9 is no longer valid. Matalas and Langbein (1962) present an alternative expression for the variance of a regional mean where the interstation correlation is included. The estimate of the regional mean is given by

$$\bar{x} = \left(\sum_{i=1}^{n} \bar{x}\right)/n_s \tag{16}$$

where $\bar{x}_i$ is the mean for each station and n_s is the number of stations. If all the n_s stations have a common variance, σ^2, and each of the n_s series is a random series (i.e., there is no serial correlation) then the expression for the variance of the regional mean is given by

$$\mathrm{Var}(\bar{x}) = \frac{\sigma^2}{N_s n_s}\left[1 + \bar{\rho}\ (n_s - 1)\right] \tag{17}$$

where N_s = the number of observations at each station (assumed constant between stations)

$\bar{\rho}$ = the average of all the $\binom{n}{2}s$) cross-correlations.

Matalas and Langbein (1962) introduced a quantity referred to as the effective number of stations, n_e. The information contained in n_s spatially correlated stations is equivalent to that contained in n_e uncorrelated stations. The relationship is given by the equation

$$n_e = n_s\ [1 + \bar{\rho}\ (n - 1)]^{-1}. \tag{18}$$

Although this equation cannot be used directly to design monitoring networks, it does provide a means of demonstrating the importance of considering interstation correlation.

Figure 14 shows equation 18 graphically. The effect of interstation correlation on computing a regional mean is obvious. It can be seen that even at very low correlation coefficients the effective number of uncorrelated stations is reduced dramatically. From equation 18 it is seen that if $\bar{\rho} = 0.1$ a maximum of ten effective stations exist. That is, the information obtained from an infinite number of stations with an average correlation coefficient of 0.1 is equivalent to the information obtained from ten uncorrelated stations. This does not, however, imply that only ten stations should be sampled. It only provides a comparison

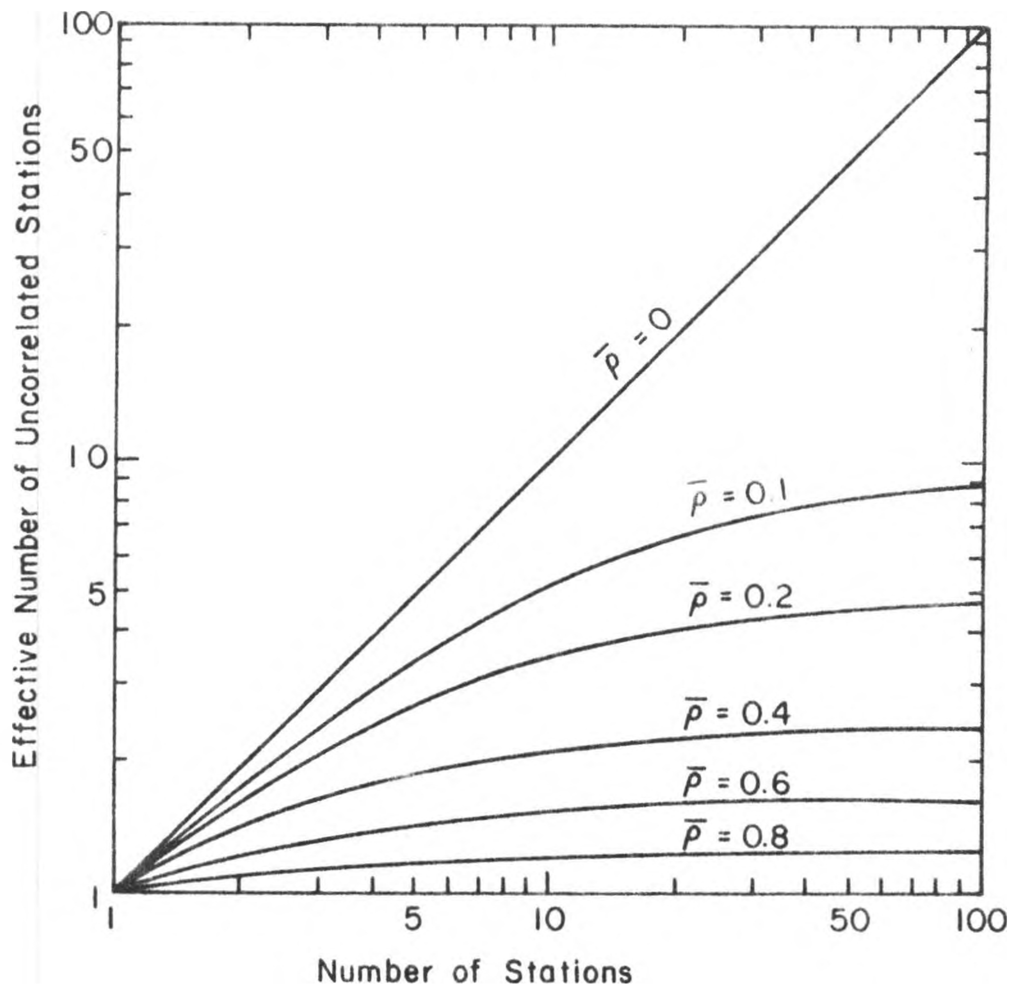

Figure 14. Effective Number of Stations in Computing Regional Means (Matalas and Langbein, 1962).

of the information being obtained relative to the information which would be obtained if the stations were actually uncorrelated.

The direct application of considering interstation correlation in designing monitoring networks is shown by the substitution of the expression for Var($\bar{x}$) from equation 17 into equation 7 to obtain the total number of samples required for a specified confidence interval about a regional mean. Using the sample estimates of variance and correlation coefficient, this results in the following equation

$$N_s n_s \geq \frac{t^2_{\alpha/2}\ s^2}{P^2}\left[1 + \bar{r}\ (n_s - 1)\right]. \tag{19}$$

Again the population variance has to be estimated from historic data or by one of the other methods previously discussed. However, in this case the average correlation coefficient has to be determined as well. Once these quantities have been established, the number of samples required at each station can be determined.

<u>Serially Correlated Observations</u>

In addition to considering spatial correlation in the development of ground water monitoring strategies, it might also be important to account for serial correlation. Haan (1977) notes that it is not uncommon to find a time series of hydrologic data that exhibits serial correlation, i.e., the observations taken at one time period are correlated with the observations taken at a previous time period.

This problem has been discussed on several studies concerning the development of monitoring strategies for surface water quality management (Loftis, 1979; Lettenmaier, 1978; Sanders and Adrian, 1978). However, it has not been addressed in the literature concerning ground water monitoring. The primary restriction in determining the effect of serial correlation, as with spatial correlation, is the lack of data. Without an adequate data base it is impossible to fully analyze the effect of any type of correlation on the final outcome of a monitoring program. Therefore, only a few general considerations will be discussed here. As more data becomes available, it will be important to determine the situations in which one needs to account for serial correlation.

A serially correlated time series can be represented by a deterministic and a stochastic component

$$x_t = y_t + z_t \tag{20}$$

where x_t = the value of the observation at time t

y_t = the value of the deterministic component at time t

z_t = the value of the stochastic component at time t.

z_t in the above expression must be stationary.

As noted by Loftis (1979), when serial correlation is considered, the variance of the sample mean is given by the expression

$$\mathrm{Var}(\bar{x}) = \frac{\sigma_z^2}{n^2}\left[n + 2\sum_{k=1}^{n-1} (n-k)\,\rho(k)\right] \tag{21}$$

where $\rho(k)$ = lag-k autocorrelation coefficient

$\sigma_z^2 = \mathrm{Var}[z_t]$.

In order to use equation 21 the values of the autocorrelation function $\rho(k)$ must be known for k = 1, 2, 3,..., n - 1. However, in order to estimate n - 1 autocorrelation coefficients, it is necessary to have considerably more than n data points.

A better approach, recommended by Loftis (1979), is to remove the deterministic component from the historic data record and fit the remaining stochastic component to a time series model. Then the theoretical autocorrelation function can be determined from this time series model. This procedure should require less data and result in an autocorrelation function which is more representative of the underlying stochastic process than an autocorrelation function estimated from a particular realization of that process. Since this type of analysis requires a more extensive data base than is currently available for ground water quality variables, a more detailed description of how this analysis would be made will not be presented here [for more details, refer to Loftis (1979)].

The necessity of considering serial correlation in establishing confidence intervals about the mean of a water quality time series will depend on the sampling frequency. If the samples are spaced far enough apart in time, the serial correlation will have a minimal effect on the confidence interval. However, if high sampling frequencies are required, serial correlation could have a very definite effect. Some results have been reported which indicate the effect of serial correlation versus sampling frequency for surface water quality variables. However, it should be noted that serial correlation would be expected to be more pronounced for ground water due to the dampening effects that exist in a ground water situation.

For the same reasons that one would expect serial correlation to be greater in ground water as compared to surface water, it would also be anticipated that the variance of a particular water quality variable would be less. For a population with a reduced variance, less sampling is required. Hence, some type of trade-off between these two quantities will exist and will be reflected in the number of samples required to obtain a certain confidence interval.

Without an adequate data base it is impossible to determine the effect of spatial or serial correlation on the development of a monitoring strategy. Therefore, it is recommended that the most logical procedure to follow is to assume independence of observations in the initial stages of the monitoring program. As more data becomes available, the validity of this assumption can be checked. More importantly, the consequences of violation of this assumption on the particular monitoring objectives can be determined and the monitoring program can be adjusted accordingly.

Sampling Techniques

A number of sampling techniques, or strategies, can be employed in an overall monitoring plan for sampling in both space and time. Therefore, it is important to consider how these techniques will function and the relative advantage s and disadvantages of each. The text on sampling techniques by Cochran (1977) was used as a basis for this discussion except where otherwise indicated. The material presented here is not intended to be an exhaustive description of all possible applications of sampling theory to ground water monitoring. Rather, the intention is to describe some of the more basic techniques that appear to have a direct application to developing monitoring strategies.

Three types of sampling will be considered: simple random sampling, systematic sampling, and stratified random sampling. Simple random sampling is a method of selecting n units out of the total population in such a way that each unit of the population has an equal probability of being accepted. Systematic sampling consists of selecting a unit at random from 1 to j and then selecting every j^{th} unit thereafter. In stratified random sampling the total population is divided into subpopulations referred to as strata. Then, a simple random sample is taken independently from each strata.

Simple Random Sampling

Simple random sampling could conceivably have application to both sampling in space and sampling in time. If samples are taken from existing wells in an aquifer, these existing wells can be considered a subpopulation of all possible locations in the basin. As such, a simple random sample taken from those wells will insure that no correlations will be introduced as a result of the sampling technique. Assuming no spatial or serial correlation the variance of the sample mean will be given as

$$\mathrm{Var}(\bar{x}) = \frac{\sigma^2}{n} \quad \frac{N - n}{N} \tag{22}$$

where N = total number of units in the population.

This is the same as equation 9 with the exception of the term $(\frac{N - n}{N})$ which is the finite population correction factor. In most cases concerning ground water monitoring, this correction factor will not be important since an infinite population is usually involved. However, in certain populations it is important. This would be the case when an entire population is considered to consist of a limited number of existing wells.

The definition of what comprises the population will vary depending on the objectives. If the purpose of monitoring is to determine the quality of drinking water, then the population consists of all wells from which drinking water is obtained. However, if the monitoring objective is to determine the average water quality for the entire aquifer, then all possible locations within the aquifer constitute the population. In this case inferences made from existing wells strictly apply to that subpopulation only. Extrapolation of such inferences to the entire basin requires the additional assumption that wells are located randomly throughout the basin.

Systematic Sampling

As previously stated systematic sampling refers to the procedure where every j^{th} unit is sampled. If a population of N units is considered and n units are to be sampled, a unit is selected at random from the first $j = \frac{N}{n}$ units. Every j^{th} unit thereafter is sampled. If N is not an integral multiple of j, some of the possible samples will have one unit less than others. Murthy (1967) recommends a procedure to avoid this problem. The N units are regarded as being arranged around a circle and j is selected as the integer nearest to $\frac{N}{n}$. A random unit is chosen from 1 to j and then every j^{th} unit is selected going around the circle until n units have been sampled.

With regard to developing monitoring strategies, the primary application of systematic sampling is sampling in time. Although simple random sampling can be used for sampling in time, from a management standpoint systematic sampling is more practical. This is particularly true if the sampling is to be frequent. The performance of systematic sampling relative to simple random sampling (or stratified sampling to be discussed later) depends largely on the structure of the population. For some populations systematic sampling is very precise while for others it is even less precise than simple random sampling. If the population itself is effectively random, systematic sampling is equivalent to simple random sampling.

Caution must be exercised in using systematic sampling with populations which exhibit period variations typical of hydrologic phenomena. The effectivenss of this type of sampling with periodic populations depends on the value of j, the interval between samples. When j is equal to or is an integral multiple of the period, systematic sampling is very inefficient. The most favorable case occurs when j is an odd multiple of the half-period. Therefore, when using systematic sampling, it is necessary to have some prior knowledge of the time series structure of the population.

Stratified Random Sampling

In stratified sampling the entire population is divided into nonoverlapping subpopulations. These subpopulations are referred to as strata and together they comprise the total of the population. If a simple random sample is selected from each stratum, the procedure is called stratified random sampling.

There are a number of reasons for using the technique of stratified sampling. One of the primary reasons is that stratification may produce gains in the precision of estimates for the population under consideration. This occurs when a heterogeneous population is divided into relatively homogeneous subpopulations and the analyses are performed on these subpopulations. Also, if it is desired to obtain data of known precision for certain subdivisions of the population, it is advisable to consider the subdivisions as separate populations. Finally, stratification may be appropriate simply because of the increased convenience of administering the sampling program.

Stratified random sampling has several applications in developing monitoring strategies. Ward et al. (1976) used stratified sampling to allocate the number of samples to stations in a surface water quality monitoring network. Montgomery and Hart (1974) note that it might sometimes be possible to use knowledge of the variability of effluent or river quality to reduce sampling frequency by stratification.

In its simplest form stratified random sampling calls for weights to be assigned to each strata. The weights for all strata must sum to one:

$$\sum_{i=1}^{L} w_i = 1 \tag{23}$$

where w_i is the weighting factor for the i^{th} station and L is the number of strata. The weight for each strata can be selected on the basis of any of several criteria. Ward et al. (1976) recommended as one possibility letting w_i represent the closeness of the i^{th} station's mean (obtained from historic data) to some standard for a particular variable. That is, a station which is close to the standard would be given more weight than one which is consistently far away from the standard. Another possibility would be to base the weighting factor on water use for a particular area. High priority uses would be given a higher weight than low priority uses.

The population mean obtained from stratified random sampling is denoted by

$$\bar{x}_{st} = \sum_{i=1}^{L} w_i \bar{x}_i \tag{24}$$

where $\bar{x}_{st}$ = sample mean of the entire population (regional mean) obtained by stratified sampling

$\bar{x}_i$ = sample mean of the i^{th} strata.

Assuming the samples are drawn independently in each strata and no spatial or serial correlations exist, the variance of the mean is given as

$$\mathrm{Var}(\bar{x}_{st}) = \sum_{i=1}^{L} w_i^2 \; \mathrm{Var}(\bar{x}_i). \tag{25}$$

Equation 25 illustrates one of the more salient features of stratified sampling. The variance of $\bar{x}_{st}$ depends only on the variances of the individual stratum means. Therefore, if the strata can be chosen such that they are relatively homogenous, the variance of the estimated population mean can be reduced significantly.

Equation 25 can also be written as

$$\mathrm{Var}(\bar{x}_{st}) = \sum_{i=1}^{L} w_i^2 \; \frac{s_i^2}{n_i} \; (1 - f_i) \tag{26}$$

where f_i represents the sampling fraction, $\frac{n_i}{N_i}$, of the i^{th} stratum. If the finite population correction factor can be omitted, then

$$\mathrm{Var}(\bar{x}_{st}) = \sum_{i=1}^{L} w_i^2 \; \frac{s_i^2}{n_i}. \tag{27}$$

The number of samples to be taken per strata can be selected to minimize the variance of the sample mean for a specified cost or to minimize the cost for a specified variance. If the cost of sampli · an be written as a linear function,

$$C = c_o + \sum_{i=1}^{L} c_i n_i \tag{28}$$

where C = total cost of sampling

c_o = overhead cost

c_i = cost per sample in stratum i

n_i = number of samples in stratum i,

then either of the minimizations given above will occur when n_i is proportional to $w_i s_i / \sqrt{C_i}$. This leads to the following general rules. In a given stratum, a larger sample should be taken if the stratum is larger, the stratum is more variable internally, or sampling in the stratum is less expensive.

If cost is fixed, the number of samples required for optimum allocation is given by

$$n = \frac{(C - c_o) \sum_{i=1}^{L} (w_i s_i / \sqrt{C_i})}{\sum_{i=1}^{L} (w_i s_i \sqrt{C_i})} . \tag{29}$$

If the cost per sample is the same in all strata, the optimum allocation for a fixed cost reduces to optimum allocation for a fixed total sample size, n. For a fixed sample size, $\mathrm{Var}(\bar{x}_{st})$ is minimized if

$$n = n \frac{w_i s_i}{\sum_{i=1}^{L} w_i s_i} . \tag{30}$$

This is sometimes referred to as Neyman allocation.

Most of the preceding discussion relates to stratification by area. It may also be advantageous to stratify by time. This will be especially true if a linear trend is present. Also, if the particular variables under consideration exhibit a significant seasonal fluctuation, stratification by seasons may result in much more reliable estimates of annual means.

Since sampling will be in a two-dimensional frame (space and time) it is quite possible that a combination of sampling techniques will provide the most effective monitoring strategy. The decision as to which techniques to use will depend on the characteristics of each individual situation.

Comparison of Sampling Techniques

One of the main objectives of a trend monitoring program is the detection of any trend over time. For populations which consist entirely of a linear trend, systematic sampling is more precise than simple random sampling. However, stratified radom sampling (with one unit per stratum) is more precise than systematic sampling. Cochran (1977) has shown mathematically that

$$V_{st} \leq V_{sy} \leq V_{srs} \tag{31}$$

where V_{st} = variance from stratified random sampling

V_{sy} = variance from systematic sampling

V_{srs} = variance from simple random sampling.

These results are rather easily demonstrated by reference to Figure 15. It can be seen that the variance for either systematic or stratified random sampling will be smaller than the variance for simple random sampling. Furthermore, the variance for systematic sampling will likely be larger than the variance for stratified random sampling. This is the case since the stratified sample allows for within-strata errors to cancel out, whereas, if a systematic sample is too low or too high in one strata, it will remain so in all strata.

For serially correlated populations it has been shown that stratified random sampling is superior to simple random sampling. No such general result can be established for systematic sampling; however, Cochran (1977) has shown that if the correlogram which represents the population is concave upward, systematic sampling is more precise than stratified random sampling. The correlogram is a plot of the autocorrelation coefficient, $\rho(k)$, versus the lag, k. One would anticipate that the correlogram would be concave upward for ground water quality

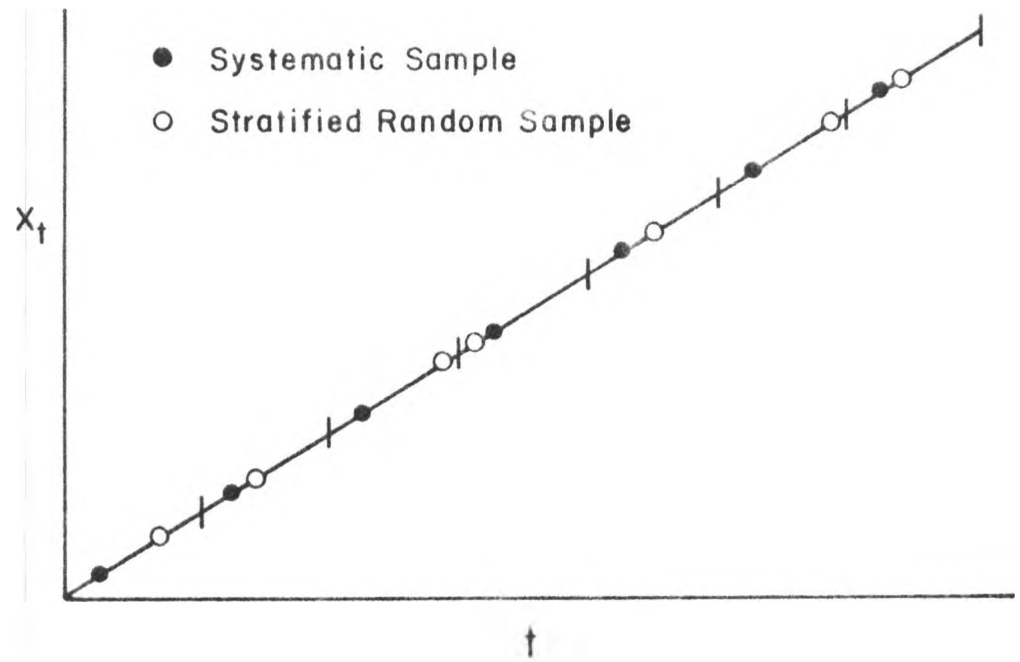

Figure 15. Sampling in a Population with Linear Trend (Cochran, 1977).

data since the autocorrelation coefficients would be expected to be positive and decrease as the time between observations increases.

Physical Aspects of Trend Monitoring

For the sake of completeness some considerations are included which relate to the physical aspects of monitoring. As noted in Chapter 3 many of these considerations apply to both inspection and trend monitoring.

The major requirement of a monitoring well is to provide representative samples of sufficient quantity at a minimal cost. Diefendorf and Ausburn (1977) note that small diameter PVC riser pipe and plastic well screens or slotted plastic pipes will generally be preferred over large diameter metal casings and screens. Besides the prohibitive cost of metal casings, anomalies in sampling could result if the ground water is reactive with metal.

Well installation should follow the same procedures used when installing water supply wells. Clean filter sand should be packed into the annulus where the screened zone will be located. However, it should be realized that monitoring wells are not intended to produce large quantities of water and costly well development procedures to remove all the fine particles of a completed well are not necessary.

Every precaution should be taken to insure that contamination does not enter the well except through the screen. The annulus should be thoroughly sealed above the screened section and backfilled to the surface. An impermeable seal should be placed at the surface. The well should remain capped when not in use.

For shallow wells samples can be collected using a simple bailer or a manual or portable pump. Wells should be pumped as slowly as possible to avoid increasing the turbidity of the sample. For deep wells a submersible pump can be used.

For most analyses only a small sample is necessary. Analysis of the samples should be considered by a qualified laboratory and approved procedures should be used for obtaining and handling the samples.

Over an extended period of time, as in trend monitoring, the cost of collecting and analyzing samples will far exceed the cost of well installation. Therefore, a management agency should make certain that correct procedures are used in installing monitoring wells. All information obtained from analysis of the data will depend on accurate and representative sampling.

CHAPTER 6

SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

Summary

This study combined deterministic and probabilistic approaches in order to develop ground water monitoring strategies for the support of community management of on-site sewage disposal systems. Two types of monitoring were discussed. Inspection monitoring was defined as the monitoring related to the inspection of an individual system to determine if it is exceeding its design discharge of pollutants to the ground water. Trend monitoring referred to the monitoring which detects over time and space the cumulative effect of all the systems in a given area.

A discussion of the variables which should be included in a monitoring program of this type was presented. It was determined that the primary variables to consider were nitrates, fecal coliforms, and in some cases viruses and phosphorus. However, the monitoring program was based on detection of nitrates alone. The rationale for this approach was also presented.

Inspection monitoring strategies were developed by means of a mathematical model which described the flow of nitrates through the leach field. The input concentrations to the leach field were simulated from data on septic tank effluent. These data were considered representative of the best practical treatment currently available from septic tank systems. Random failures and temporary overloads were also included in the input. Nitrate flow through the leach field was modeled via the convective-dispersion equation and various sampling plans were superimposed on the output in order to evaluate the effectiveness of those plans in detecting system failure.

Sensitivity analysis was conducted to determine the effect of changing physical parameters of the system, i.e., ground water depth and soil type. Also, the effect of changing input conditions was tested by considering different randomizations of the input failures and temporary overloads.

Results of the model were used to determine the sampling frequency required to obtain a certain probability of detection for each of the sampling plans evaluated. Sampling location was also discussed. It was recommended that the results of an inspection monitoring program be used in conjunction with mass balance calculations in order to determine the pollution effect on the aquifer.

The development of trend monitoring strategies consisted primarily of applying existing statistical theory to the problem of ground water quality monitoring. A brief review of previous studies regarding ground water monitoring was presented along with a discussion of the application of those studies to the monitoring of on-site disposal systems. This was followed by a discussion of the number of samples required to obtain a specified confidence interval about the estimated mean of a water quality variable. The effect of spatial and serial correlation was also considered. Finally, a discussion of various sampling techniques and their application to ground water monitoring was presented.

Conclusions

1. The most important variables to incorporate in a ground water monitoring program to support community management of on-site disposal systems are nitrate and fecal coliforms. If the disposal system is immediately adjacent to a body of surface water, it may be necessary

to include phosphorus. In locations where the underlying strata consist of fractured bedrock, monitoring for viruses may important.

2. In developing an inspection monitoring program, the mathematical model indicated the physical characteristics of the leach field (i.e., soil type and ground water depth) are not critical to failure detection. (This assumes that the sample is always taken near the water table surface). However, the time from system failure to detection of that failure varies considerably depending on soil type.
3. The variability of the input to the leach field is critical to the operation of a sampling plan with regard to detecting failure. In the initial development of a monitoring strategy, it will be necessary to make assumptions regarding this input and the probability of failure of a particular system. As the data base increases, changes in the monitoring strategy can be made if the data analysis indicates it is necessary.
4. The definition of failure, used in the inspection monitoring model, is an extreme event failure. Where average concentrations being discharged from a leach field are more relevant to the chosen management strategy, the statistical procedures described for trend monitoring should become the basis for designing the inspection monitoring system.
5. A monitoring program designed to detect temporary system overloads would require sampling frequencies in excess of that which could be considered feasible for a management agency.
6. It is recommended that an inspection monitoring program should be directed toward quantifying the output of individual systems at the

bottom of the leach field (the same as the water table in this study). The results from such a monitoring program can be used in mass balance calculations to determine the pollution effect on the aquifer.

7. The results given in this report can be utilized to decide which sampling plan and sampling frequency to use for a specific situation if the input to the leach field is similar to that assumed in this study.
8. For complete management of on-site systems it will be necessary to implement both inspection monitoring and trend monitoring.
9. In the initial stages of developing a trend monitoring program, it will be difficult to assess the effects of spatial and serial correlations unless some preliminary data are available. Given this fact, it is recommended that the assumption of independent observations be made. As more data are collected, the validity of this assumption can be examined and the monitoring strategy altered if necessary.
10. Several sampling techniques (e.g., simple random sampling, stratified random sampling, and systematic sampling) could be used effectively in ground water monitoring for sampling in both space and time. However, in order to determine which technique would be most advantageous in a given situation, it will be necessary to have some knowledge of the characteristics of the population being sampled.

Recommendations

The model from which inspection monitoring strategies were developed was necessarily hypothetical. Consequently, an important recommendation for future study is the field verification of the results obtained from this model. However, any research effort of this nature should be directed toward verification of the results in terms of monitoring objectives rather than simple verification of the model itself.

Since it has been determined that input to the leach field has a very definite effect on monitoring strategies, a more exact quantification of this input would serve to better establish appropriate monitoring strategies. In particular, a precise definition of the type of failure which can be expected from a septic tank leach field system is needed.

More study regarding the population characteristics of ground water quality variables will allow better evaluation of available sampling techniques. As monitoring programs are initiated, this should be viewed as a primary objective. With careful planning, an iterative procedure can be followed in developing monitoring strategies. As more data becomes available, the monitoring program can be reevaluated and altered if necessary. Such a procedure would result in continual improvement of the monitoring program and increased reliability of the results obtained.

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APPENDIX

COMPUTER PROGRAM LISTING

List of Variables Used in Computer Program

ARG1	=	error function argument from convective-dispersion equation (unsaturated case)
ARG2	=	error function argument from convective-dispersion equation (saturated case)
AVGT	=	average time from failure to detection, days
C1	=	concentration calculated at the water table, mg/ℓ
CDETEC	=	detection concentration, mg/ℓ
CO1	=	input concentration set randomly from normal distribution for background concentration and set equal to COVRLD for temporary overloads and system failure, mg/ℓ
CO2(M)	=	concentration used as input to saturated zone, mg/ℓ
CONC	=	concentration at sampling point, mg/ℓ
CONEW	=	change in input concentration, mg/ℓ
COVRLD	=	overload concentration, mg/ℓ
D1	=	unsaturated dispersion coefficient, cm^2/day
D2	=	saturated dispersion coefficient, cm^2/day
DISP	=	subroutine which calculates convection-dispersion for unsaturated and saturated zones
DTIME	=	time from system failure to detection, days
DUM	=	dummy variable used for argument in RANSET function
DUR(K)	=	duration of temporary overload determined randomly on interval (0,3)
GWDPTH	=	depth to ground water, cm
ITRIAL	=	index which identifies particular simulation run
J	=	number of changes in input concentration
N	=	RN converted to integer
NFAIL	=	number of failure detections

NJ	=	flag variable set equal to 0 if successive temporary overloads do not overlap, set equal to 1 if they do overlap
NM	=	index for TEST, number of times temporary overloads overlap
NNODET	=	number of no detections
NSAMPL	=	number of primary samples taken per year
NTAU1	=	variable used to set TAU1(J) after completion of a temporary overload
NTEMP	=	number of temporary overload detections
NTRIAL	=	number of simulation runs
PFAIL	=	percent failure detection
PNODET	=	percent no detection
PTEMP	=	percent temporary overload detection
QUANT1	=	1-erf(ARG1), unsaturated case
QUANT2	=	1-erf(ARG2), saturated case
RANDEV	=	random deviate from normal distribution with given mean and standard deviation
RANF	=	intrinsic function used to generate random numbers
RANSET	=	intrinsic function used to initialize random number generator
RANU	=	random number determined on the interval (0,1)
RCONC	=	subroutine which sets input concentrations
RMEAN	=	mean concentration of the input nitrate, mg-N/ℓ
RN	=	number of temporary system overloads determined randomly on the interval (1,50)
RTIME	=	subroutine which determines random time for system failures, random number of temporary overloads, and random times and durations of overloads
SDEPTH	=	depth below ground water at which sample was taken, cm
STDEV	=	standard deviation of input nitrate, mg-N/ℓ
STIME(I)	=	day on which sample is taken
SUM	=	sum of times from system failure to detection, days

SYFAIL	=	time of system failure, days
T	=	increment of time for variation in background concentration
TAU(I)	=	time of temporary overload determined randomly on interval (0,365) days
TAU1(J)	=	time of change in input concentration, days
TAU2(M)	=	time of change in concentration at the water table, days
TEMP	=	temporary storage location used for rearranging TAU
TEST	=	test variable to determine if TAU(I) + DUR(I) extends to SYFAIL or beyond (temporary overloads were only considered up to the time of system failure)
TEST(NM)	=	test variable used to determine the duration of overlapping temporary overloads
TSET	=	subroutine which sets the time of each change in inflow concentration as required by subroutine DISP
TT	=	current time, days
V1	=	unsaturated seepage velocity, cm/day
V2	=	saturated seepage velocity, cm/day
Y	=	random number on interval (0,1)

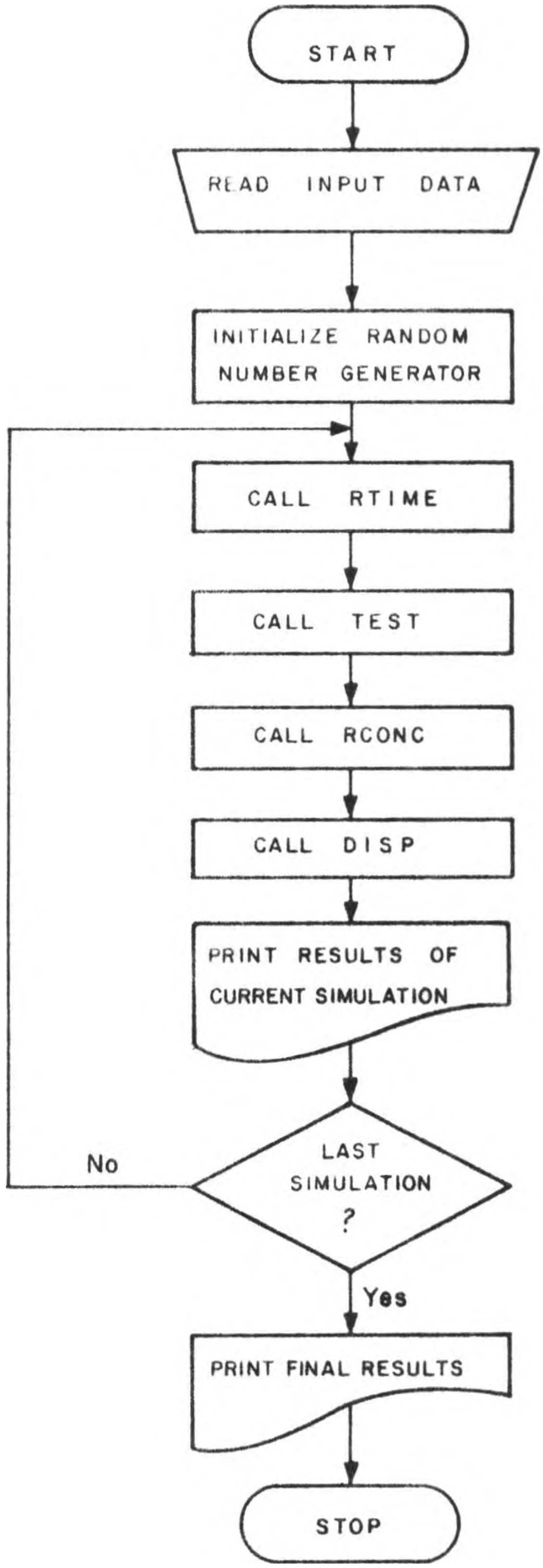

Figure A1. Flow Chart for Program SAMPL.

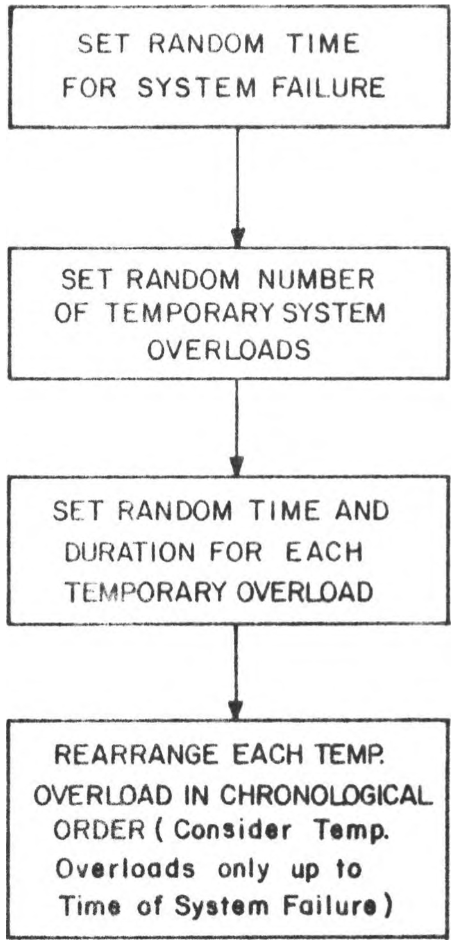

Figure A2. Flow Chart for Subroutine RTIME.

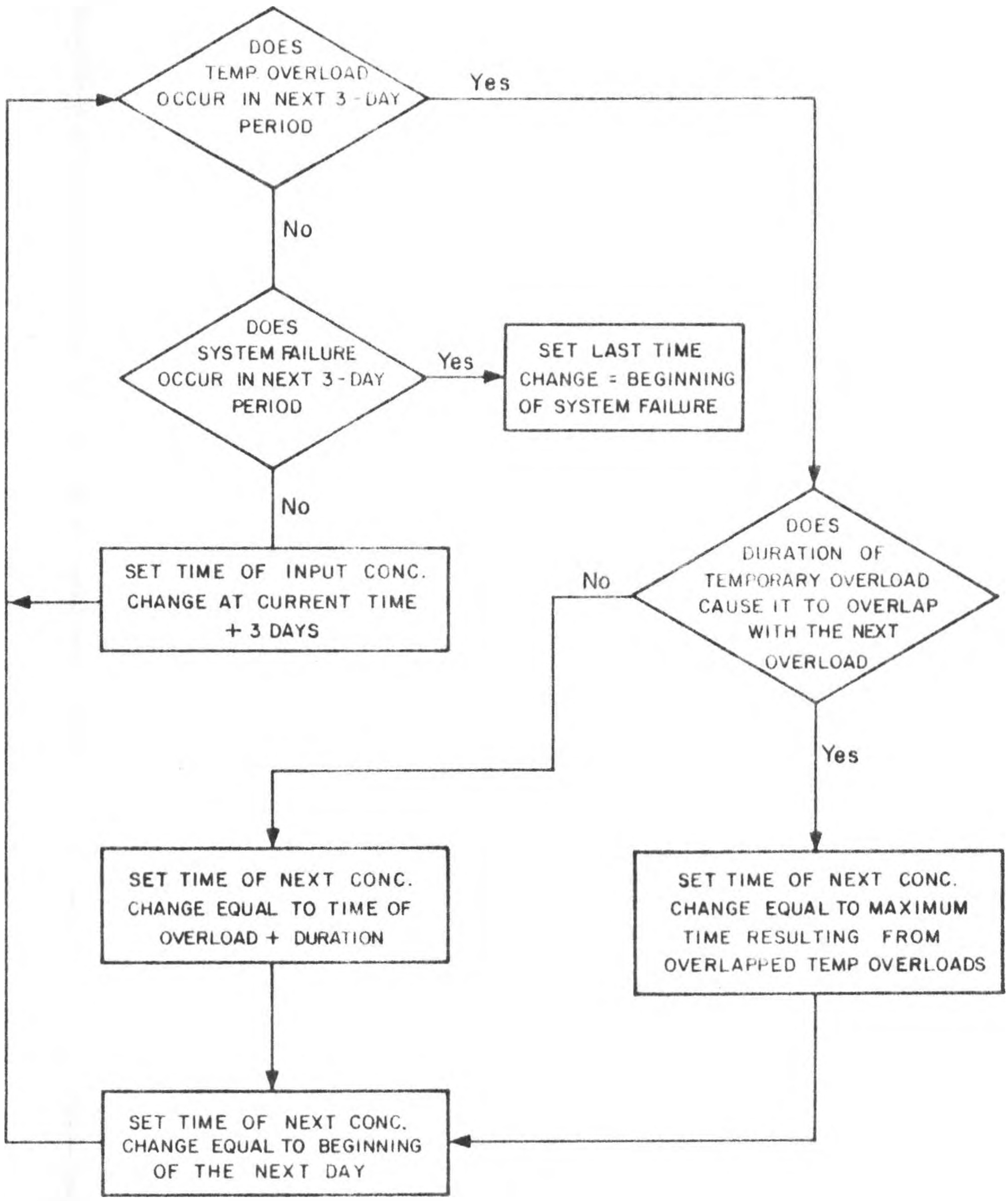

Figure A3. Flow Chart for Subroutine TSET.

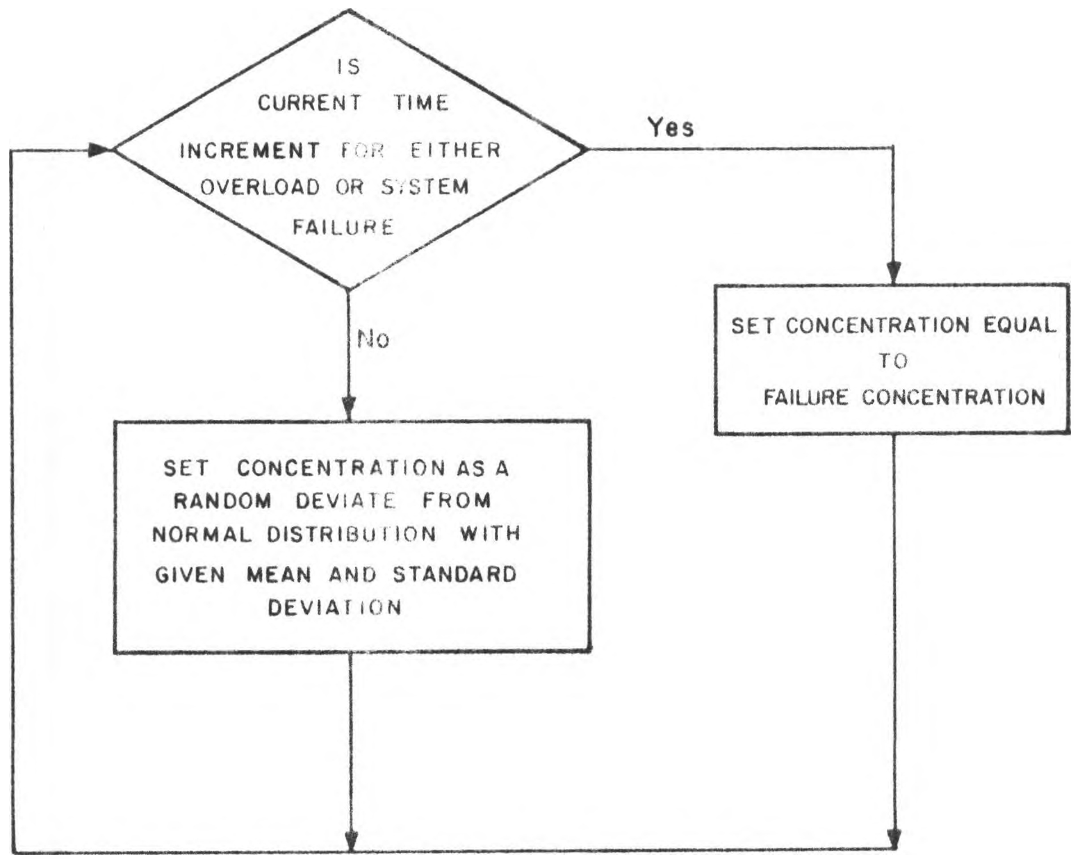

Figure A4. Flow Chart for Subroutine RCONC.

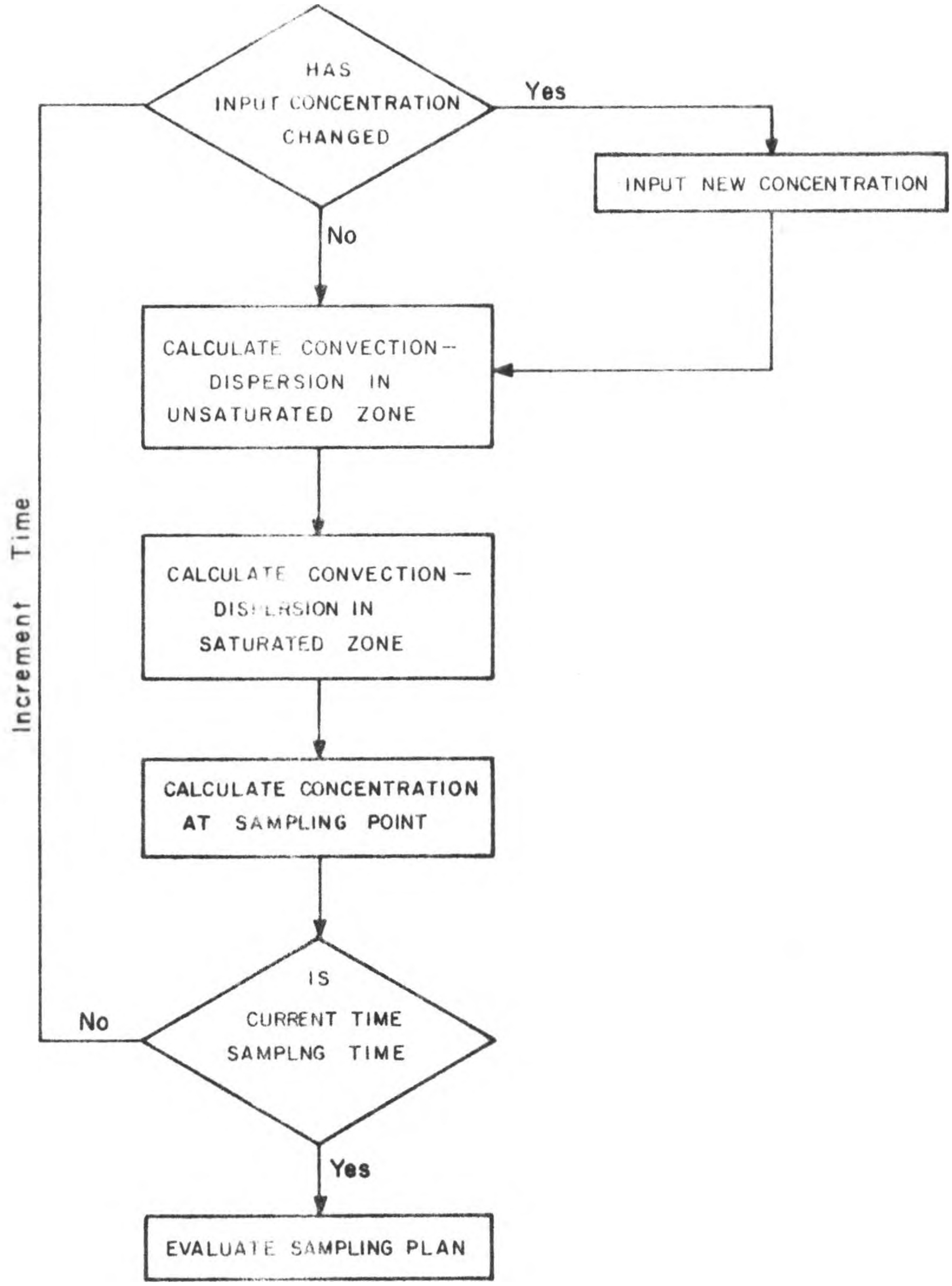

Figure A5. Flow Chart for Subroutine DISP.

```
      PROGRAM SAMPL(INPUT,OUTPUT,TAPE5=INPUT,TAPE6=OUTPUT)
C
C  PROGRAM SAMPL DETERMINES THE EFFECTIVENESS OF A PARTICULAR SAMPLING PLAN IN
C  DETECTING FAILURE OF A SEPTIC TANK - LEACH FIELD SYSTEM.  EFFECTIVENESS IS
C  MEASURED BY THE PROBABILITY OF FAILURE DETECTION AND THE LENGTH OF TIME FROM
C  SYSTEM FAILURE TO DETECTION.
C
      DIMENSION TAU(150), DUR(150),TAU1(500),CO1(500),STIME(100)
C  READ AND PRINT INPUT DATA.
      WRITE(6,150)
  150 FORMAT("0","INPUT DATA")
      READ(5,100) V1,D1,V2,D2
  100 FORMAT(4F10.2)
      WRITE(6,151) V1,D1,V2,D2
  151 FORMAT(1X,"UNSAT. VEL. =",F8.2," CM/DAY",5X,"UNSAT. DISP. COEFF. =
     $",F7.2," CM**2/DAY"/1X,"SAT. VEL. =",F8.2," CM/DAY",5X,"SAT. DISP.
     $ COEFF. =",F7.2," CM**2/DAY")
      READ(5,101) GWDPTH
  101 FORMAT(F10.1)
      WRITE(6,152) GWDPTH
  152 FORMAT(1X,"DEPTH TO GROUNDWATER =",F6.1," CM")
      READ(5,102) COVRLD,CDETEC
  102 FORMAT(2F10.1)
      WRITE(6,153) COVRLD,CDETEC
  153 FORMAT(1X,"OVERLOAD CONC. =",F6.1," MG/L",5X,"DETECTION CONC. =",F
     $6.1," MG/L")
      READ(5,103) SDEPTH,NSAMPL
  103 FORMAT(F10.1,I5)
      WRITE(6,154) NSAMPL
  154 FORMAT(1X,"NUMBER OF PRIMARY SAMPLES PER YEAR =",I3)
      READ(5,106) RMEAN,STDEV
  106 FORMAT(2F10.1)
      WRITE(6,164) RMEAN,STDEV
  164 FORMAT(1X,"MEAN CONC. =",F6.1,5X,"STAND. DEV. =",F6.1)
      WRITE(6,165)
  165 FORMAT(1X,"PRIMARY SAMPLING DAYS")
      DO 10 I=1,NSAMPL
      READ(5,104) STIME(I)
  104 FORMAT(F10.1)
      WRITE(6,155) STIME(I)
  155 FORMAT(1X,F10.1)
   10 CONTINUE
      READ(5,105) NTRIAL
  105 FORMAT(I5)
      WRITE(6,156) NSAMPL
  156 FORMAT("0","SAMPLING PLAN -- ",I3," PRIMARY SAMPLES PER YEAR. IF T
     $HE CONCENTRATION IS GREATER THAN OR EQUAL TO DETECTION CONC., TWO"
     $,/10X,"SECONDARY SAMPLES ARE TAKEN 3 DAYS AND 6 DAYS LATER.  TWO P
     $OSITIVE SECONDARY SAMPLES CONSIDERED SYSTEM FAILURE.")
      WRITE(6,157) SDEPTH
  157 FORMAT(1X,"SAMPLE DEPTH = ",F4.1,"CM BELOW GROUNDWATER DEPTH")
      WRITE(6,158)
  158 FORMAT("0",8X,"TRIAL",9X,"TEMPORARY OVERLOAD DETECTION",9X,"FAILUR
     $E DETECTION",9X,"TIME FROM FAILURE TO DETECTION (DAYS)")
C  INITIALIZE VARIABLES.
      NFAIL = 0
      NTEMP = 0
      NNODET = 0
      SUM = 0.0
C  INITIALIZE SEED FOR RANDOM NUMBER GENERATOR VIA CLOCK TIME.
      DUM = CLOCK(0.)
C  CALL RANDOM NUMBER GENERATOR.
      CALL RANSET(DUM)
C  RUN SIMULATION NTRIAL TIMES.
      DO 1 ITRIAL=1,NTRIAL
      DUM = DUM*1.082946
      CALL PTIME(DUM,TAU,DUR,N,SYFAIL)
      CALL TSET(N,TAU,DUR,TAU1,1)
      CALL RCONC(J,DUM,STDEV,RMEAN,CO1,N,TAU1,TAU,COVRLD,DUR)
      CALL DISP(V1,D1,V2,D2,GWDPTH,CDETEC,SYFAIL,STIME,SDEPTH,TAU1,CO1,
     $NFAIL,NTEMP,NNODET,ITRIAL,AVCT,J,NSAMPL,SUM)
```

```
    1 CONTINUE
      WRITE(6,159) NTRIAL
  159 FORMAT ("0","NUMBER OF TRIALS = ",I4)
C  CALCULATE AND PRINT PERCENTAGE OF FAILURE DETECTION,TEMPORARY OVERLOAD
C  DETECTION, NO DETECTION AND THE AVERAGE TIME FROM FAILURE TO DETECTION.
      PFAIL = NFAIL
      PTEMP = NTEMP
      PNODET = NNODET
      RTRIAL = NTRIAL
      PFAIL = (PFAIL/RTRIAL)*100.0
      PTEMP = (PTEMP/RTRIAL)*100.0
      PNODET = (PNODET/RTRIAL)*100.0
      WRITE(6,160) PFAIL
  160 FORMAT(1X,"PERCENT FAILURE DETECTION = ",F5.1)
      WRITE(6,161) PTEMP
  161 FORMAT(1X,"PERCENT TEMPORARY OVERLOAD DETECTION = ",F5.1)
      WRITE(6,162) PNODET
  162 FORMAT(1X,"PERCENT NO DETECTION = ",F5.1)
      WRITE(6,163) AVGT
  163 FORMAT(1X,"AVERAGE TIME TO DETECTION (DAYS) = ",F5.1)
      STOP
      END



      SUBROUTINE RTIME(DUM,TAU,DUR,N,SYFAIL)
C
C  THIS SUBROUTINE DETERMINES RANDOM TIMES FOR SYSTEM FAILURE AND OVERLOADS ON
C  THE INTERVAL (0,365) AND RANDOM DURATIONS OF THE OVERLOADS ON THE
C  INTERVAL (0,3)
C
      DIMENSION TAU(150),DUR(150)
C  DETERMINE A RANDOM TIME FOR SYSTEM FAILURE ON THE INTERVAL (0,365) AND
C  DETERMINE A RANDOM NUMBER OF TEMPORARY SYSTEM OVERLOADS FROM (1,50)
      DO 10 KK=1,2
      RANU = RANF(0)
      IF(KK.EQ.2) GO TO 11
      SYFAIL = RANU*365.0
      GO TO 10
   11 RN = RANU*40.0+1.0
   10 CONTINUE
      N = RN
C  DETERMINE A RANDOM TIME FOR EACH TEMPORARY OVERLOAD ON THE INTERVAL (0,365)
      DO 12 I=1,N
      RANU = RANF(0)
      TAU(I) = RANU*365.0
   12 CONTINUE
C  DETERMINE A RANDOM DURATION FOR EACH TEMPORARY OVERLOAD ON THE INTERVAL (0,3)
      DO 13 K=1,N
      RANU = RANF(0)
      DUR(K) = RANU*3.0
   13 CONTINUE
C  REARRANGE TAU(I) IN INCREASING ORDER
      NA = N-1
      DO 15 J=1,NA
      M = J
      MA = J+1
      DO 14 I=MA,N
   14 IF (TAU(I).LT.TAU(M)) M=I
      TEMP = TAU(J)
      TAU(J) = TAU(M)
   15 TAU(M) = TEMP
C  ONLY CONSIDER TEMPORY OVERLOADS UP TO THE TIME OF SYSTEM FAILURE
      DO 16 I=1,N
   16 IF (TAU(I).GE.SYFAIL) GO TO 17
   17 N = I
      TAU(N) = SYFAIL
C  IF TAU(I)+DUR(I) EXTENDS TO SYFAIL OR BEYOND,SET SYFAIL AT THE VALUE OF
C  TAU(I).
```

```
18 I = N-1
   TEST = TAU(I)+DUR(I)
   IF (TEST.LT.TAU(N)) GO TO 19
   N = I
   GO TO 18
19 TAU(N) = SYEATI
   RETURN
   END


   SUBROUTINE TSET(N,TAU,DUR,TAU1,J)
C
C  THIS SUBROUTINE USES THE RANDOM VALUES OF TAU AND DUR AND SETS THE TIME OF
C  EACH CHANGE IN INFLOW CONCENTRATION AS REQUIRED FOR SUBROUTINE DISP.
C
   DIMENSION TAU(N),DUR(N),TAU1(500),TEST(50)
   TAU1(1) = 0.0
   T = 3.0
   J = 2
   I = 1
   M = 1
   NM = 1
   NJ = 0
   DO 40 K=1,N
   IF (NJ.EQ.1) GO TO 46
41 IF (TAU(K).LE.T) GO TO 45
   TAU1(J) = T
   J = J+1
   T = T+3.0
   GO TO 41
45 TAU1(J) = TAU(K)
   IF (K.EQ.N) GO TO 40
   J = J+1
C  IF SUCCESSIVE VALUES OF TAU+DUR OVERLAP SET TAU1 AT THE LARGEST VALUE OF
C  TAU+DUR.
46 TEST(NM) = TAU(K)+DUR(K)
   KK = K+1
   IF (TEST(NM).GT.TAU(KK)) GO TO 35
   IF (NM.EQ.1) GO TO 11
   DO 10 II=2,NM
10 IF (TEST(II).GT.TEST(M)) M=II
11 TAU1(J) = TEST(M)
   M = 1
   NM = 1
   NJ = 0
   NTAU1 = TAU1(J)+1.0
   J = J+1
   TAU1(J) = NTAU1
   IF (TAU(KK).GT.TAU1(J)) GO TO 47
   T = TAU1(J)
   GO TO 40
47 T = TAU1(J)+3.0
   J = J+1
   GO TO 40
35 NJ = 1
   NM = NM+1
40 CONTINUE
   RETURN
   END
```

```
      SUBROUTINE RCONC(J,DUM,STDEV,RMEAN,CO1,N,TAU1,TAU,COVRLD,DUR)
C
C  THIS SUBROUTINE SETS CO1 AT COVRLD FOR EACH TIME INCREMENT BEGINNING WITH
C  TAU(TIME OF RANDOM OVERLOAD OR SYSTEM FAILURE.)  FOR ALL OTHER TIME
C  INCREMENTS, CO1 IS GIVEN A RANDOM VALUE FROM A NORMAL DISTRIBUTION (MEAN =
C  RMEAN, STANDARD DEVIATION = STDEV.)
C
      DIMENSION CO1(J),TAU1(J),TAU(N),DUR(N)
      K = 1
      DO 100 I=1,J
      IF (TAU1(I).NE.TAU(K)) GO TO 50
   40 M = K
      K = K+1
      TEST = TAU(M)+DUR(M)
      IF (TAU(K).LT.TEST) GO TO 40
      CO1(I) = COVRLD
      GO TO 100
C  SELECT A RANDOM DEVIATE FROM THE STANDARD NORMAL DISTRIBUTION AND CONVERT IT
C  TO A RANDOM DEVIATE FROM THE NORMAL DISTRIBUTION WITH THE GIVEN MEAN AND
C  STANDARD DEVIATION (REFERENCE -- HILLIER AND LIEBERMAN,OPERATIONS RESEARCH).
   50 A = 0.0
      DO 75 M=1,12
      Y = RANF(0)
   75 A = A+Y
      RANDEV = (A-6.0)*STDEV+RMEAN
      IF (RANDEV.LT.0.0) RANDEV=0.0
      CO1(I) = RANDEV
  100 CONTINUE
      RETURN
      END


      SUBROUTINE DISP(V1,D1,V2,D2,GWDPTH,COEFFC,SYFAIL,STIME,SDEPTH,TAU1
     $,CO1,NFAIL,NTEMP,NNODE1,ITRIAL,AVGT,J,NSAMPL,SUM)
C
C  THIS SUBROUTINE CALCULATES CONVECTION-DISPERSION OF THE CONTAMINANT AS IT
C  MOVES THROUGH THE SOIL.  THE INPUT TO THE UNSATURATED ZONE IS THE NITRATE
C  CONCENTRATION BELOW THE CRUSTED LAYER (ASSUMING COMPLETE NITRIFICATION HAS
C  OCCURED).  THE INPUT TO THE SATURATED ZONE IS THE CALCULATED CONCENTRATION
C  AT THE UNSATURATED-SATURATED INTERFACE.
C
      DIMENSION TAU1(J),CO1(J),TAU2(1000),CO2(1000),STIME(100)
      NTIME = 365
      IJ = 1
      TAU2(1) = 0.0
      II = 2
      L = 1
      M = 1
      NC = 0
      DO 20 III=1,NTIME
      II = III
C  THIS PORTION OF THE SUBROUTINE CALCULATES CONVECTION-DISPERSION IN THE
C  UNSATURATED ZONE.  THE CONCENTRATION AT THE UNSATURATED-SATURATED INTERFACE
C  IS COMPUTED.
      V = V1
      D = D1
      C1 = 0.0
      X = GWDPTH
      IF (L.EQ.J) GO TO 5
      IF (II.LT.TAU1(II)) GO TO 5
      L = L+1
      II = II+1
    5 CONTINUE
      DO 30 I=1,L
      T = II-TAU1(I)
      IF (T.EQ.0.0) GO TO 31
      ARG1 = (X-V*T)/(2.0*SQRT(D*T))
      QUANT1 = 1.0-ERF(ARG1)
      CONEW = CO1(I)
      IF (I.EQ.1) GO TO 11
```

```
      K = I-1
      CONEW = CO1(I)-CO1(K)
   11 CONTINUE
      CONC = (CONEW/2.0)*QUANT1
   30 C1 = C1+CONC
   31 CONTINUE
C   THIS PORTION OF THE SUBROUTINE CALCULATES CONVECTION-DISPERSION IN THE
C   SATURATED ZONE.  THE CONCENTRATION CALCULATED AT THE GROUND WATER DEPTH IS
C   USED AS INPUT CONCENTRATION.  TIMES OF INPUT CONCENTRATION CHANGES ARE
C   CALCULATED.  CONCENTRATION AT THE SAMPLING DEPTH IS COMPUTED.
      V = V2
      D = D2
      CO2(M) = C1
      IF (M.EQ.1) GO TO 12
      TAU2(M) = TT
      JJ = M-1
      IF (CO2(JJ).NE.CO2(M)) GO TO 12
      M = JJ
   12 CONTINUE
      C2 = 0.0
      X = SDEPTH
      T = TT-TAU2(I)
      IF (T.EQ.0.0) GO TO 41
      ARG2 = (X-V*T)/(2.0*SQRT(D*T))
      QUANT2 = 1.0-ERF(ARG2)
      CONEW = CO2(I)
      IF (I.EQ.1) GO TO 13
      KK = I-1
      CONEW = (CO2(I)-CO2(KK))
   13 CONTINUE
      CONC = (CONEW/2.0)*QUANT2
   40 C2 = C2+CONC
   41 CONTINUE
      M = M+1
C   IF CURRENT TIME IS A SAMPLING TIME, EVALUATE THE SAMPLING PLAN.
      IF (TT.NE.STIME(IJ)) GO TO 20
      IF (C2.LT.CDETEC) GO TO 33
      NC = NC+1
      IF (NC.EQ.3) GO TO 75
      STIME(IJ) = STIME(IJ)+3.0
      GO TO 20
   75 CONTINUE
      IF (STIME(IJ)-SYFAIL) 15,14,14
   14 DTIME = STIME(IJ)-SYFAIL
      STIME(IJ) = STIME(IJ)-6.0
      NFAIL = NFAIL+1
      SUM = DTIME+SUM
      AVGT = SUM/NFAIL
      WRITE(6,50) ITRIAL,DTIME
   50 FORMAT(9X,I4,23X,"NO",20X,"YES",32X,F5.1)
      GO TO 10
   15 WRITE(6,51) ITRIAL
   51 FORMAT(9X,I4,22X,"YES",30X,"NO",32X,"-----")
      STIME(IJ) = STIME(IJ)-6.0
      NTEMP = NTEMP+1
      GO TO 10
   33 IJJ = IJ
      IF (NC.EQ.0) GO TO 76
      IF (NC.EQ.2) GO TO 77
      STIME(IJ) = STIME(IJ)-3.0
      GO TO 76
   77 STIME(IJ) = STIME(IJ)-6.0
   76 CONTINUE
      IJ = IJ+1
      NC = 0
      IF (IJJ.LT.NSAMPL) GO TO 20
      WRITE(6,52) ITRIAL
   52 FORMAT(9X,I4,23X,"NO",30X,"NO",32X,"-----")
      NNODET = NNODET+1
      GO TO 10
   20 CONTINUE
   10 CONTINUE
      RETURN
      END
```