UNC-WRRI-96-304

EVALUATION OF AEROBIC PRETREATMENT/LAND DISPOSAL SYSTEMS FOR ON-SITE WASTEWATER DISPOSAL

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The research on which this report is based was financed by the United States Department of the Interior, Geological Survey, through the N.C. Water Resources Research Institute.

WRRI Project No. 70115
Agreement No. 14-08-0001-G2037
USGS Project No. 07 (FY '92)
September 1996

One hundred fifty copies of this report were printed at a cost of $1254.00 or $8.36 per copy.

ACKNOWLEDGMENT

The research reported here could not have been completed without cooperation of a number of individuals who dedicated their time unselfishly and/or provided materials with no expectation. Of particular we owe a great deal of debt to Mr. Kevin Martin, President of Soil and Environmental Consultants, for his involvement in all aspects of this project, from the initial stages of site selection to design and installation of the experimental systems and their monitoring. Special gratification is also extended to the North State Utilities and Harco Construction Company for allowing access to their septic system at Manchester Subdivision in North Wake County and to the homeowner of our site in Chatham County. We like to thank the members of the Division of Environmental Management and On-Site Wastewater Section of the North Carolina Department of Environment, Health, and Natural Resources, Mr. Everett Lynn from Wake County Health Department, and Mr. Jimmy Collins from the Chatham County Health Department for their cooperation with the project. Special thanks to Waste Water Systems, Inc., for allowing the use of their equipment, and particularly to Mr. Tom Sinclair and his staff for their assistance in installing the drip experimental system at our Wake County site. We also express our appreciations to Mr. Jerry McKinney, from Clearstream, for providing an aerobic treatment unit, pumps, and drip lines at discount, and for his help and that of Mr. Truett Kastner for installing the system at the Chatham County site. We also extend our appreciations to Dr. Robert (Bob) Rubin and Mr. L.T. Woodlief from the Biological and Agricultural Engineering Department at North Carolina State University for their cooperation in installing the drip system at the Wake County site. Others that need special recognition are Dr. Joseph Kleiss, Christopher Niewoehner, Brian Smith, Chester Cobb, Thomas Boyce, Steve Price, Elizabeth Morey, Ed Surbrugg, Kanglin Li, Edwin Stott, Chris Wooten, Tim Sudderth, and Chris Bordeaux, all current or past members of faculty, staff and students in the Soil Science Department at NCSU. Like all other projects, our jobs were made easier because of the cooperation of secretarial and accounting staff at both the Water Resources Research Institute and the Soil Science Department. For that, we extend our sincere gratitude to them. Finally, we would like to express our appreciations to all research staff and faculty in the Soil Science Department and all other individuals who helped us in our efforts, and apologize for not mentioning them by name.

ABSTRACT

The increase in North Carolina population in the last decade has been accompanied by a larger demand for housing in rural areas and around major municipalities with no access to a public sewage system. Septic tank-soil absorption systems provide a viable option for managing domestic sewage; however, not all the soils in North Carolina are suitable for septic systems. The general objective of this research was to evaluate low-pressure pipe (LPP) and drip systems for the disposal of treated and untreated wastewater and assess a spray irrigation system for surface disposal of treated effluent by an aerobic treatment unit (ATU).

At a site of a community septic system serving a number of homes in a subdivision in Wake County, experimental septic systems, composed of four LPP and four drip disposal subfields, were installed for subsurface disposal of untreated effluent from the community septic system and for treated effluent from an ATU that was installed for the study. Loading rates of 0.075, 0.1, and 0.125 gal ft^{-2} d^{-1} for the LPP and 0.1 and 0.2 gal ft^{-2} d^{-1} for drip systems were evaluated at this site. At a second site in Chatham County with unsuitable soils for subsurface disposal of wastewater, a septic system composed of an ATU and spray irrigation system was selected for evaluation. A small experimental drip system was installed adjacent to the spray field at this site using two loading rates of 0.05 and 0.1 gal ft^{-2} d^{-1}. At both sites, the drip lines were installed in trenches dug by a trenching machine and backfilled with soil without any gravel. Intact and disturbed samples were collected from various depth/horizons at both sites and analyzed for various physical and chemical properties. In situ saturated hydraulic conductivity (K_{sat}) of various horizons was also measured at both sites. Tensiometers were installed at different depths inside and outside the trenches of the LPP and the drip systems for measuring soil water pressure head, and time domain refelectometry (TDR) was used to measure soil water content outside the trenches from the soil surface to 60 cm depth. The distribution of wastewater over the spray field was also assessed.

The soils at the Wake county site showed considerable variation with respect to depth and particle size distribution within small areas for the LPP and drip systems. Wastewater ponding was observed more frequently in the trenches of the LPP system receiving untreated effluent. Higher soil water content and pressure head were measured around the trenches receiving untreated wastewater than the ones receiving treated effluent from the ATU. Overall, observation of wastewater ponding in the trenches of the LPP system, and assessment of soil water pressure head and water content at various depths indicated that during the study period wastewater treated by the ATU infiltrated the soil and moved away from the LPP or drip lines at a faster rate than the untreated effluent. At the Chatham County site, the soils in the spray irrigation and drip system areas had a relatively thin A horizon underlain by a thick, very sticky, very plastic Bt horizon. Overall, wastewater distribution over the spray irrigation field was not uniform. However, due to low loading rate not exceeding 1.1 cm per week (0.45 in wk^{-1}) and relatively permeable surface horizon with high infiltration capacity, the amount of runoff from the spray irrigation field was not generally quantifiable. As for the experimental drip system, the trenches for the drip lines were dug into the Bt horizon, and because of extremely low conductivity, and perhaps lateral movement of rainwater infiltrating the area above the drip lines, ponding was observed in the trenches for most of the study period while no wastewater was

observed at the soil surface. Evaluation of the rate of flow of tap water from orifices of the two drip systems installed above ground for part of the study indicated that the flow rate from individual orifices of each system is fairly uniform when the system is fully pressurized. However, lack of uneven distribution during the early stages of water application, as well as during drainage indicated the need for a better design and installation of drip systems to minimize overloading certain parts of their respective drainfield areas.

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SUMMARY AND CONCLUSIONS

Increased demand for housing development in unsewered areas of North Carolina has increased the need to develop new technologies and evaluate present alternative septic systems for on-site treatment and disposal of household wastewater. The available alternatives include pretreating the septic tank effluent by individual aerobic treatment units (ATUs) and disposing the effluent using low-pressure pipe (LPP), drip, or spray irrigation system. Because the ATUs and drip systems are relatively new in North Carolina, little information is available about their performance and allowable loading rates for various soils.

This study was conducted to assess the effect of pretreatment of septic tank effluent by an ATU on the performance of low-pressure pipe (LPP) and drip systems in a soil suitable for septic systems, and to evaluate the performance of a spray irrigation and a drip system for the disposal of wastewater treated by an ATU in a soil considered unsuitable for subsurface disposal of household wastewater.

Four small LPP subfields, each containing three lateral lines, were installed in the repair area of a large community septic system serving 17 homes in a subdivision in the north part of Wake County. The soil at the site was considered suitable for a subsurface septic system with a minimum loading rate of 0.1 gal ft^{-2} d^{-1} (4 L m^{-2} d^{-1} or 0.4 cm d^{-1}). Two of the subfields were assigned to receive untreated effluent from the main pump tank of the large community LPP septic system and the other two were assigned to receive effluent treated by an ATU that was installed at the site for this study. Three loading rates, 0.075, 0.1, and 0.125 gal ft^{-2} d^{-1} (3, 4, and 5 L m^{-2} d^{-1}, respectively) were applied to the three drainlines in each subfield. Wastewater ponding in the trenches, soil water content adjacent to the trenches, and soil water pressure head under and adjacent to the trenches were monitored weekly using observation wells, time domain reflectometry (TDR), and tensiometry, respectively.

To study drip systems, four small subfields (each containing a total of 63 m of lateral lines placed in three parallel, 10-cm wide and 20- to 40-cm deep trenches dug with a trenching machine) were installed in the buffer zone of the community septic system at the Wake County site. The drip lateral lines were placed in the trenches, and the trenches were backfilled with soil with no gravel around the drip lines. To monitor the lines, an observation well was installed in one of the trenches of each subfield before backfilling the trenches. at one location within each subfield. Two of the subfields received untreated wastewater from the main pump tank at the rates of 0.1 and 0.2 gal ft^{-2} d^{-1} (4 and 8 L m^{-2} d^{-1}, respectively) and the other two received treated effluent from the ATU at the above rates. Wastewater level in each observation well and soil water pressure head around one emitter and adjacent to one of the drip lines in each subfield were monitored weekly. The soil profile at the site was described by hand-auger boring and feel textural analysis technique. Saturated hydraulic conductivity and other soil parameters were evaluated in situ and in the laboratory using intact and bulk soil samples. Wastewater samples were collected from the drip and LPP subfields and analyzed for BOD_5, COD, and coliform bacteria.

At another site located in Chatham County, an existing septic system composed of a septic tank, an ATU, a chlorinator, a storage tank (also known as pump tank), and a spray field serving a three-bedroom home was selected for study. In this system, wastewater from the septic tank was treated by the ATU and disinfected with chlorine before being applied to the spray field. A small drip system (a different brand than the one used at the Wake County site) was installed below the spray irrigation field. The drip system was composed of five 15-m long laterals installed inside 15-cm wide and 30- to 45-cm deep trenches without any gravel. Three of the lateral lines received wastewater (treated and disinfected) at the rate of 0.05 gal ft^{-2} d^{-1} (2 L m^{-2} d^{-1}) and two of the lines received wastewater at 0.1 gal ft^{-2} d^{-1} (4 L m^{-2} d^{-1}). Observation wells, TDR, and tensiometry were used to monitor saturation in and outside the trenches, soil water content outside the trenches, and soil water potential at various depths and locations, respectively. For the spray irrigation field, the uniformity of wastewater application was monitored by placing 44 cans (15-cm diameter) on a predetermined grid pattern to cover a full circle and two half circles of three adjoining spray heads. The soil at the site was characterized in situ, and intact and bulk samples were collected for laboratory analyses. Runoff samples from the spray field were collected and analyzed for pH, electrical conductivity (EC), nitrate, and ammonium. Wastewater samples from the septic tank, the ATU (before chlorination), the pump tank (after chlorination), and from the drip field were also collected and analyzed for BOD_5, COD, and coliform bacteria.

Using small above ground system and tap water, the performance of each brand of drip systems was assessed by measuring the discharge from individual orifices on the drip lines under a prescribed pressure. For this purpose, the drip lines for each brand were installed on a totally flat plane (zero slope along the lines), a 1.25% uniform slope along the drip lines, and with slight differences in the elevation of the orifices along the drip lines. Following application of tap water under a recommended pressure for each system, the volume of water flowing from individual orifices was measured for three different time periods. The time periods were immediately after application of water to the system, when the system was fully pressurized and all orifices on the lines were actively discharging water, and immediately after water application to the system was ceased (free drainage period).

The soils at the Wake County site varied considerably within the small LPP and drip systems. On the average, the Bt horizon in the upper LPP subfield extended from 30 to 127 cm depth, whereas in the lowest subfield, the Bt horizon was rather thin, and saprolite was observed at 70 to 90 cm below the soil surface. The maximum clay content in the Bt horizon was 46.2% for the upper part and 70.4% for the lower part of the experimental LPP area. Similarly, in the subfields of the drip systems, the thickness of Bt and depth to saprolite varied considerably, but the maximum clay content remained above 60%. In general, the lowest saturated hydraulic conductivity (K_{sat}) in both LPP and drip subfields was measured in the transitional BC horizons. The low K_{sat} of the BC was responsible for impeding water movement below the drainfield areas.

Wastewater ponding was observed frequently in the trenches of the LPP system receiving untreated effluent. For the trenches receiving treated effluent, wastewater ponding was observed only after major rainfall events. Soil water pressure head values for under the trenches of the subfields receiving treated and untreated effluent were near zero for most of the study period. The soil water pressure head at various depths outside the trenches corresponded well with the ponding of wastewater in the trenches as well as with the thickness of the Bt and depth to the slowly permeable BC horizon. The soil water content values for different depth intervals from the soil surface to 60 cm depth for the LPP subfields receiving treated effluent were less than the corresponding values for the subfields receiving untreated wastewater. Observation of wastewater ponding in the trenches, the soil water pressure head values, and soil water content data indicated that during the study period wastewater treated by the ATU infiltrated into the soil and moved away from the trenches at a faster rate than the untreated effluent.

For the drip systems, higher level of wastewater ponding was observed more in the observation wells installed in the backfilled trenches of the subfields receiving untreated effluent than the ones receiving treated wastewater. Presence of liquid in the observation wells can be taken as an indication of saturated conditions around the drip lines. Local saturation around the drip lines was perhaps due to a lack of uniform distribution of wastewater over the entire area of each subfield. Assessment of the soil water pressure head at various depths indicated that for the duration of the monitoring treated wastewater moved away from the drip lines at a faster rate than the untreated wastewater. Overall, the quality of wastewater treated by the ATU with respect to BOD_5, COD, and fecal coliform was similar to the values reported in the literature. In this study, wastewater with substantially less BOD_5 and COD appeared to be moving from the trenches and through the soil solum and its underlying strata at a faster rate than the untreated wastewater. The filter system on the drip systems used at the Wake County site was effective in lowering the BOD_5 and COD, but had less effect on the coliform bacteria.

At the Chatham County site, the soil in the spray field area had a thin A horizon underlain by a 70-cm thick, very sticky, very plastic Bt horizon. The C horizon had a massive structure and was firm, sticky and plastic. In the drip field area, the depth to Bt ranged from 12 cm in the upper part of the field to as much as 30 cm at the bottom part of the field which was near a natural drainage. In both spray and drip fields, bulk density of the soil below the Ap horizon was greater than 1.57 g cm^{-3}, indicating a tightness that could restrict downward water movement. Saturated hydraulic conductivity measured in situ and in the laboratory using intact core samples was generally low for the Bt horizon. Because of a low-conductivity Bt horizon, wastewater combined with infiltrating rainwater entered the backfilled trenches of the drip system, and the observation wells in the upper part of the drainfield remained partially full for an extended period of time. Since no gravel was used around the dip lines in the original trenches, we assume that the Bt horizon in the upper part of the drainfield remained saturated as long as wastewater was observed in the observation wells. Less ponding than in the upper part of the drainfield was observed in the observation well located in the lowest part of the drainfield with 30 cm depth to the Bt and close proximity to a natural drainage. The soil water pressure head under the trenches and outside the trenches corresponded fairly well with the ponding in the observation wells (i.e., saturation around the drip lines) and the restrictive nature of the Bt horizon. Once the Bt horizon was wetted, it prevented downward movement of wastewater applied by the drip and/or spray system as well as the natural precipitation during the time when evapotranspiration was low. The soil water content data also agreed well with the soil profile characteristics in the upper and lower parts of the drip field. Since the Bt horizons in the types of soil found at this site do not drain easily, the soil below the drainfield area remained relatively wet during the entire study period.

Wastewater distribution over the spray irrigation field was not uniform. However, due to a low loading rate (1.1 cm or 0.45 in per week), and relatively permeable surface layers, the amount of runoff from the spray field was not generally quantifiable. Overall, the nonuniformity of wastewater distribution was due to the drifting of the applied wastewater by the prevailing wind across the spray field as well as the distribution and growth of the vegetative cover, which was trees and underbrush at this site. The quality of runoff collected from the site was within acceptable limits and did not appear to pose any serious danger. Although aerobic treatment of wastewater by the ATU seemed to substantially improve the wastewater quality, the type of chlorinator used at this site did not appear to function at a desirable level.

Measurements of tap water flow from individual orifices on the drip lines of the above ground systems indicated that the flow rate from the orifices of each system was fairly uniform and near its design value when the entire system was pressurized. However, measurements during the initial period of water application (pressurizing period) and after ceasing water application to each system (final free drainage) showed that neither of the systems used in our study was able to distribute water uniformly over an area. Lack of uniform distribution was due to the lag time between flow from the orifices at the beginning of the drip system and flow from the orifices at the end of the system when the system was started, as well as to the free drainage that occurred from some orifices when water application was stopped.

Overall, it appears that treating the septic tank effluent by an ATU (or perhaps other treatment systems such as a sand filter) has a positive effect on the movement of wastewater from the trenches of the LPP and drip systems into the soil and away from the drainfield area of the respective system. Drip systems are viable options for wastewater disposal in shallow soils as well as for soils with adequate depth for conventional septic systems. Drip systems, however, must be installed on contour to reduce the potential of nonuniformity of wastewater application. More studies are needed to assess the potential use of drip systems in soils that have a relatively shallow top soil with relatively high saturated hydraulic conductivity but with a slowly permeable subsoil. Also, the potential use of surface drip systems for disposal of treated and disinfected wastewater in North Carolina needs to be studied. A spray irrigation system is suitable for wastewater disposal provided that wastewater is treated and disinfected adequately. Transport of aerosols from spray irrigation systems and the potential of reducing the required buffer zone around the spray irrigation fields should be studied to determine if spray irrigation systems can be used to dispose of septic tank effluent on lots that are 2 to 4 acres.

RECOMMENDATIONS

1. Variability of the thickness of horizons above the transitional horizon(s) in the Piedmont region must be evaluated over the entire area considered for the drainfield of a septic system. The depth to the least permeable layer and its permeability must be determined for proper selection of a wastewater disposal system. This is particularly important for large septic systems where the drainfield is generally composed of a number of subfields that are used individually in series or other patterns.

2. Similar to LPP systems, the individual lateral lines of drip systems must be installed on contour lines. Precautions should be taken in designing and installing the drip systems to minimize the free drainage from the orifices or to force equal drainage from individual orifices or sections of the drip lines after stopping wastewater application to the system. In addition, installation of drip lines with uniform slope along the lateral lines must be avoided.

3. Extreme care should be taken during installation of drip lines in trenches. Although the experimental systems in our study were not installed by a vibratory plow, it appears that such installation is advantages over the trench installation technique. The trench system may provide large pockets where wastewater can become ponded, particularly in soils that have slowly permeable subsoil material.

4. Blockage of orifices of the drip systems with solids present in septic tank effluent or build-up of organic material over or inside the orifice mechanism must be evaluated. For those systems that require no pretreatment, the filter system should be checked to assure that the amount of solids entering the lateral lines is minimal. In addition, the long term performance of automated and manual flushing of the lateral lines must be fully evaluated. For those systems requiring pretreatment, both the pretreatment unit and the filter mechanism in the drip system should be inspected regularly.

5. Use of drip systems for surface disposal of wastewater, particularly in forested areas with leaf litter should be assessed in North Carolina. Specifically, the potential for freezing of the lateral lines for surface drip systems should be determined.

6. Aerobic treatment units must be inspected regularly and serviced as needed. Disinfection systems, particularly simple chlorination systems that use chlorine tablets in the outlet of the pretreatment unit, must be evaluated to determine their effectiveness with time. A better way of providing contact time between the wastewater and chlorine tablets appears to be necessary.

7. The spray irrigation is a viable option for surface application of treated wastewater in areas where soils are considered unsuitable for septic systems with subsurface disposal. Aerosol transport from spray irrigation fields and the potential for reducing the buffer zone requirement around the spray fields should be evaluated.

8. In areas where soils are suspected of not conducting water or wastewater at a reasonable rate, and/or providing adequate treatment to allow proper functioning of a septic system (e.g.,

seasonally high water table conditions), pretreatment must be required if subsurface disposal is to be allowed. This is particularly true if saprolite with certain characteristics (e.g., shallow depth to bedrock, high clay content) is going to be permitted for the ultimate disposal of wastewater.

9. A long term (5- to 10-year) study should be conducted to assess the long term effects of pretreating wastewater by ATU and other individual wastewater treatment systems on the movement of wastewater through the soil in and around the drainfield areas of septic systems receiving treated effluent.

INTRODUCTION

The North Carolina population increased by more than 12.5% during the 1980's (Bureau of Census, 1983, 1993a). This population increase has been accompanied by an equally higher demand for the use of septic systems for on-site management of household wastewater. Although the percentage of population using on-site wastewater treatment/disposal systems (slightly more than 50%) did not change significantly from 1980 to 1990, the total number of septic systems in North Carolina increased by 30%. In 1990, the number of all housing units in the state was 2,818,193, of which 1,403,033 units were connected to a public sewer system, 1,365,632 units used septic systems (or cesspool which is not permitted in North Carolina), and the remaining 49,528 units used other means (not specified) for sewage disposal (Bureau of Census, 1993a). Nationally, based on the census data (Bureau of Census, 1993b), approximately 25% of all housing units use on-site wastewater disposal. Overall, North Carolina ranks third in the number of on-site wastewater disposal systems. The two states that rank above North Carolina (Florida and New York) have more than 1.59 and 1.51 million on-site systems, respectively, but their populations of approximately 13 and 18 million are mostly served by public sewer systems (data obtained from the National Small Flow Clearinghouse, West Virginia Univ., Morgantown, WV, 1994). Projections on the annual increase in the number of on-site systems obtained from the census data and the number of permits issued in a 6-year period from 1982 to 1987 (Hoover and Amoozegar, 1988) indicate that the number of on-site systems in North Carolina should now exceed 1.5 million.

The demand for housing development and lack of funds for public sewer systems will undoubtedly increase the use of on-site wastewater treatment/disposal systems. Construction in unsewered areas of North Carolina has used most of the suitable soils for septic systems in all three major physiographic regions of the state. As a result, a greater need for development and testing of innovative and alternative systems for on-site treatment and disposal of household wastewater has emerged.

The most common type of septic system in North Carolina is a conventional septic system composed of a septic tank and a drainfield (Hoover and Amoozegar, 1988). The trenches for conventional septic systems are generally 3 ft (0.9 m) wide with an average distance of 6 ft (1.8 m) between any two neighboring trenches. Sewage from the dwelling served by the septic system flows by gravity into a septic tank where solids are settled (equivalent to a primary treatment). Each time sewage enters the septic tank, an approximately equal volume of wastewater from the septic tank simultaneously flows into one or more trenches by the force of gravity. In this type of wastewater application to the drainfield, wastewater always enters the beginning of the trench(es) in unequal doses. As a result, an uneven distribution of wastewater within the trenches occurs, and the drainfield area of the septic system may not be utilized efficiently.

One major requirement for soil/site suitability for a septic system is the presence of at least 12 in (30 cm) of suitable soil material between the bottom of the trenches and any restrictive layer that may exist in the drainfield area. Seasonally high water table, bedrock, or saprolite with selected characteristics are some of the restrictive conditions that may render a site unsuitable for a septic system. According to the Laws and Rules for Sewage Treatment and Disposal Systems (DEH, 1993), a minimum of 36 inches (90 cm) of suitable or provisionally suitable soil materials is required for a site to be considered acceptable for installing a septic system. The majority of the soils in North Carolina, however, may not meet this requirement. Although alternative systems, such as the low-pressure pipe (LPP) system (Carlile, 1979, Cogger et al., 1982; EPA, 1980) or shallow placement (Hoover and Amoozegar, 1988), have been developed to overcome some of the problems associated with soil depth, the need for more alternative systems for treatment and disposal of domestic wastewater is greater than ever.

The low-pressure pipe septic system has been used in areas with shallow soil to a restrictive layer. This technology has been employed for various size septic systems ranging from single family homes to large community systems, as well as for returning biologically treated ground water to the water table at contaminated sites for ground water and/or soil remediation. For LPP septic systems, wastewater from the dwelling moves by gravity into a septic tank where most solids are settled. Then, wastewater containing dissolved and fine suspended solids moves by gravity into a storage tank (also known as the pump tank) where an electrical pump is installed for intermittently applying wastewater to the drainfield area. The drainfield area is composed of an array of perforated polyvinyl chloride (PVC) pipes (referred to as laterals or lateral lines) installed in a gravel envelope in narrow (15 to 30 cm wide) and shallow (30 to 60 cm deep) trenches. Depending on the design of the system, the size of the holes (perforations) on the laterals generally varies from 1/8 to 7/32 in (3.2 to 5.6 mm, respectively). The spacing between the holes on the laterals, and the distance between the centers of two neighboring trenches are generally 5 ft (1.5 m). Cogger et al. (1982) present a detailed description for the design of LPP distribution systems, including the selection of the hole size(s) and distribution system for sloping areas.

Depending on the design of the LPP system, the pump in the pump tank of the system is activated by a regular timer or a computerized system, a float system, or a combination of the timer and floats for delivering wastewater to the perforated pipe network. Application (pumping) of wastewater to the drainfield is also referred to as "dosing", and the period during which no wastewater is applied to the drainfield area is referred to as "resting cycle".

Theoretically, by varying the size of the holes and the distance between the holes and/or trenches, a LPP system can be designed to achieve a desired distribution pattern over the drainfield area of the septic system. Otis et al. (1974), Bouma (1975), and White et al. (1985) used above ground LPP distribution systems to assess the flow of water from individual holes on the laterals when the system was fully pressurized (i.e., when all the laterals were full and water was flowing out of all the holes). According to Otis et al. (1974) and Bouma (1975), a uniform distribution is achieved with LPP system installed on a level ground, and White et al. (1985) reported that the difference between the predicted and actual flow from laterals was less than 12% in their evaluation. Evaluation of the performance of LPP distribution in a number of septic systems (Martin, 1987; Weymann, 1989) and an above ground LPP distribution system (Amoozegar et al., 1994), however, have shown that the designed wastewater distribution may not be achieved under field conditions. Masking of the perforations on the laterals by gravel inside the trenches, uneven elevation along laterals in and between the trenches, air entrapment inside the laterals, partial to total blockage of the holes from inside by suspended solids, and root masking or intrusion into the holes are conditions that can result in a substantial change in the rate of flow of wastewater from individual holes after the system is installed and operated.

In recent years, drip irrigation systems, which were originally designed for irrigating agricultural crops in arid and semi-arid regions (Hall, 1985), have been adapted for wastewater disposal (Oron et al., 1991). At present, drip systems are commercially available for wastewater disposal even in the relatively humid regions of the United States.

In irrigated agriculture, the time and quantity of water applied through a drip irrigation system depends on a number of factors including the needs of the crop(s) and the soil type. With a drip irrigation system, water is generally applied in the vicinity of the root zone at such a rate that evaporation from the surface and deep percolation are minimized. The use of drip systems for wastewater disposal, however, should not be considered an irrigation system because, for most septic systems, wastewater application to the drainfield is a daily operation and is independent of the water needs of the plants that may be grown over the drainfield area. Therefore, wastewater drip systems may not operate in the same way as irrigation drip systems.

In general, a drip system for wastewater disposal is composed of a network of small (approximately ½-in) diameter flexible plastic tubing, with small orifices on 2-ft (60 cm) intervals, attached to a pumping system. The drip lines can be installed below the soil surface in narrow (e.g., 10 cm wide) and shallow (e.g., 15 cm deep) trenches dug by a trencher and covered with soil without placing any gravel in the trenches, or buried in slits dug by a vibratory plow. Because no gravel is used for installing drip lines, the installation procedure may be short compared to the installation of a conventional or LPP system. In general, the spacing between the drip lines is 2-ft (60 cm), and there is little storage capacity around the drip lines except the pore spaces of the natural soil. Wastewater from a holding tank is pumped into the drip lines under a relatively high pressure of 10 to 70 psi (equivalent to 7 to 50 m of pressure head) at regular time intervals, or at intervals based on the rate of wastewater entering the septic system.

Because drip systems are relatively new for wastewater disposal, little information is available regarding their operation and performance under different field conditions. Using observation of crops receiving wastewater from a drip system, Oron et al. (1991) indicated, without elaboration, that wastewater application by subsurface drip systems is uniform.

Unlike conventional septic systems, the common LPP and drip systems have little to no storage capacity for wastewater. Wastewater applied to the trenches of an LPP system or to the soil around the drip lines of a drip system must infiltrate the soil and move away from the drainfield area within individual dosing cycles. Similar to the conventional septic systems, wastewater accumulation in the trenches of a LPP system or in the soil around the emitters of a drip system results in prolonged saturation of the soil and may lead to formation of a biological clogging mat and perhaps surface failure of the system. Therefore, proper performance, including the uniformity of wastewater application and movement of soil water away from the drainfield area of the system, as well as management of these systems are very important.

The size of a septic system drainfield is generally based on the amount of wastewater that can be applied to a unit area of the soil under consideration (referred to as the loading rate) as well as the daily flow rate of wastewater. In North Carolina, the design loading rates for septic systems are determined primarily by the soil texture. For conventional septic systems the loading rates are based on the cumulative surface area of the bottom of the trenches, while for LPP and drip systems the loading rates are calculated based on the total area of the drainfield.

A septic tank-soil treatment system (i.e., a septic system) is a relatively simple land-based wastewater disposal system that uses the soil for the removal of harmful chemicals and biological constituents of septic tank effluent. Untreated septic tank effluent is applied to the vadose or unsaturated zone (below the natural surface or a mound constructed on top of the soil) to receive treatment. It is believed that soils with their complex chemical, physical, and microbiological characteristics can remove the pollutants present in the wastewater and allow the treated wastewater to move away from the septic tank trenches. The type of treatment expected to occur in the vadose zone includes filtration of suspended solids by the soil matrix, attenuation of dissolved chemicals or compounds by soil particles, precipitation/transformation of certain compounds, breakdown of solid particles, and destruction of bacteria and viruses. Although wastewater can receive partial treatment in the saturated zone (i.e., below a water table), presence of an aerobic zone where only a portion of soil pores are filled with water (i.e., vadose zone) is essential for a more efficient treatment. When a soil is determined to lack the potential for providing adequate treatment (e.g., sandy soil over a shallow water table), pretreatment may be required to substantially reduce the chemical and biological hazard of the wastewater. Pretreatment, followed by disinfection, is also required for surface application of wastewater (spray irrigation) in areas where soils are not suitable for subsurface disposal of wastewater.

A number of individual systems (e.g., sand filter, peat filter), with or without disinfection, can be used to pretreat household wastewater before disposal (EPA, 1980; Perley, 1985; Weymann, 1989; Hoover et al., 1991, Piluk and Peters, 1994; Osesek et al., 1994). One of these options for pretreating household wastewater before disposal is an individual aerobic treatment or Aerobic Treatment Unit (ATU) (Carlile, 1994). A few years ago, the North Carolina Laws and Rules for Sewage Treatment and Disposal Systems were amended to accept the ATU systems that meet Class I wastewater quality standards established under Standard 40 of the National Sanitation Foundation (NSF) when assurances of ongoing operation and maintenance are provided. The rules allow for some reductions in drainfield size and horizontal and vertical separation requirements when wastewater is treated by an approved ATU. In general, wastewater generated by sand filters and ATUs must be disposed of via a surface (spray irrigation) or subsurface (e.g., LPP, drip irrigation) system.

Due to the concern over transportation of pollutants by aerosols from spray irrigation fields, a relatively large buffer zone around the spray field is required (DEM, 1993). This requirement, however, will limit the number of land parcels that can utilize spray irrigation for the disposal of household wastewater. The potential of using drip irrigation systems for disposal of household wastewater below the soil surface (even at relatively low rates) will generally require less land than a comparable spray irrigation system.

Objectives

The specific objectives of the study reported here were:

1. To evaluate the long-term acceptance rate by a soil considered suitable for septic systems of wastewater treated by an ATU in conjunction with low-pressure pipe disposal.

2. To assess the performance of spray irrigation for surface disposal and drip (trickle) irrigation for subsurface disposal of treated effluent in a soil considered unsuitable for a subsurface septic system, and compare the performance of drip systems receiving treated and untreated wastewater in a soil considered suitable for septic systems in the Piedmont region of North Carolina.

MATERIALS AND METHODS

Two sites, one in northern Wake County and one in eastern Chatham County, were selected for the study. The site in northern Wake County was located in the Manchester Subdivision where 17 homes surrounding a small pond were served by a large community LPP septic system managed by a public utility company. At this site, LPP systems and drip systems receiving treated and untreated septic tank effluent were evaluated. Hereafter, this site will be referred to as the Wake County site. The site in Chatham County was a 3-bedroom, single family home located on the east side of the Triassic Basin near SR 1742. The existing septic system for the home consisted of a septic tank, an ATU, a chlorinator, a pump tank, and a spray field. At this site, the spray irrigation system and a small experimental drip system receiving ATU-treated effluent were studied. Hereafter, this site will be referred to as the Chatham County site.

Wake County Site

The community septic system serving 17 of the homes in the Manchester Subdivision was composed of a number of LPP subdrainfields covering an approximately 4,150 m^2 (approximately one acre) area and was operated by a public utility company. Wastewater from individual septic tanks at each of the 17 homes was pumped into a large central pump tank adjacent to the drainfield area of the system. Appropriate permissions were obtained from the Division of Environmental Management (North Carolina Department of Environmental Health and Natural Resources) and the public utility company to install small LPP systems in the repair area of the main system, to install small drip systems in the buffer zone area adjacent to the repair area, and to use wastewater from the main pump tank of the community system.

Before installing any of the experimental systems, the soils in the repair area and the buffer zone were evaluated by the hand-auger boring technique to determine their suitability for wastewater disposal and to determine a loading rate for the systems. A 6- by 25.5-m (20- by 85-ft) area in the repair area was selected for installing LPP systems for receiving treated and untreated effluent (Fig. 1). A 4.5- by 90-m (15- by 300-ft) area in the buffer zone next to the area for the LPP system was selected for installing the drip systems for treated and untreated effluent. These systems will be discussed in more detail.

A NSF approved ATU (Clearstream Wastewater Systems, Inc., Beaumont, Texas) constructed of fiberglass, and a 1,200-gallon concrete pump tank (marked as 2nd pump tank in Fig. 1) were installed between the area selected for the experimental systems and the main pump tank of the community system (see Fig. 1). The tank and ATU were installed side by side in a large pit dug by a backhoe. After connecting the ATU to its associated pump tank for gravity flow and testing them for leaks, they were covered with their access ports opened to the land surface. One submersible, effluent-rated pump was installed in each of the pump tank of the main septic system and the 1,200-gallon pump tank after the ATU. The submerged pump installed in the main pump tank and a solenoid valve activated by a multichannel control panel were used to transfer wastewater to the ATU for generating an adequate amount of treated wastewater for daily application to the LPP and drip systems assigned to receive treated wastewater. The same

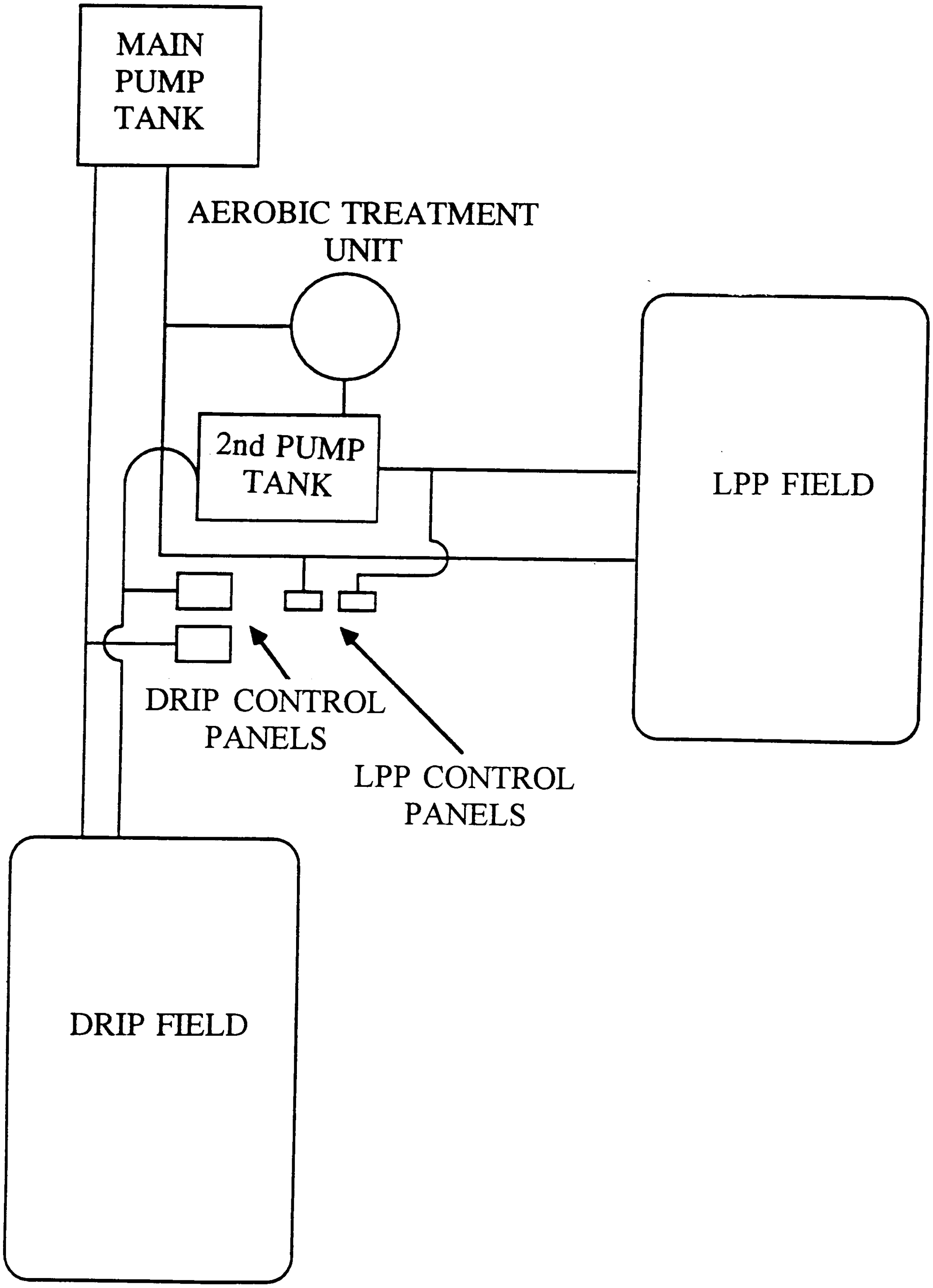

Figure 1. Schematic diagram of the plan view of the experimental systems showing the relative locations of the main pump tank, aerobic treatment unit and its associated pump tank, and the drainfield areas for the low-pressure pipe and drip systems and their respective control units.

control panel and pump were used in conjunction with a series of solenoid valves for daily application of effluent from the pump tank of the main system to the LPP subfields assigned to receive untreated effluent. Treated wastewater generated by the ATU was first transferred by gravity into the 1,200-gallon pump tank. Another multichannel control panel, a series of solenoid valves, and the pump installed in this tank were used for daily application of treated wastewater by the ATU to the respective LPP subfields.

Low-pressure Pipe Systems

For installing two small LPP systems in the repair area of the community septic system for treated and untreated wastewater, an experienced private contractor was hired to dig the trenches for the 12 laterals and their respective supply lines. According to our design, each LPP system was composed of two subfields, each with three 7-m (22-ft) long lateral lines (for a total of 12 laterals). The trenches for the laterals were placed on contour, and they were 25 cm (10 in) wide and 43 to 48 cm (17 to 19 in) deep. Our initial evaluation of the site by the hand-auger boring technique had indicated a fairly uniform soil over the experimental drainfield area. Upon inspection of the trenches and additional hand-auger borings, however, it was determined that the soil in the upper part of the experimental area was deeper to the Bt horizon than in the lower part of the field. Based on the characteristics of the upper soil horizons, the four small subfields were assigned to receive treated wastewater from the ATU and untreated wastewater from the main pump tank of the community septic system as shown in Fig. 2.

After the trenches were dug, the loose soil and debris at the bottom of each trench were removed and the bottom of the trench was leveled using a long straight edge and a hand-held level. A series of observation wells and access ports were installed in each trench before 15 to 20 cm (6 to 8 in) of gravel was placed inside the trench. After leveling the top of the gravel in each trench, a 6-m (20-ft) long lateral with four pre-drilled 5/32-in (0.4-cm) diameter holes on 150-cm (5-ft) spacing and equipped with an end turn-up was placed over the gravel with holes facing down. Two manifolds, one for carrying treated wastewater and one for untreated effluent were placed in the trenches between the upper two and the lower two subfields as shown in Fig. 2. Each lateral was then connected to its respective manifold through a flow meter, a solenoid valve, and a gate valve as shown in Fig. 3.

Three different loading rates were selected for each of the subfields of the two LPP systems. A loading rate of 0.1 gal ft^{-2} d^{-1} (4 L m^{-2} d^{-1}), determined from the respective range of loading rates for the type of the soil at the site (DEH, 1993), was chosen as the main design loading rate for the systems. Loading rates of 0.125 gal ft^{-2} d^{-1} and 0.075 gal ft^{-2} d^{-1} (5 and 3 L m^{-2} d^{-1}, respectively) were used for the other two loading rates (25% above and 25% below the main loading rate, respectively). The pressure head for all 12 laterals was set at 90-cm (3-ft). The respective dosing rate for each line was obtained by adjusting the dosing time during which the solenoid for the line was opened for wastewater application to the trench. As mentioned earlier, two separate multichannel irrigation control panels were used for application of the treated and untreated wastewater to the LPP subfields. The flow meter for each lateral line was used to monitor the amount of wastewater application to each trench.

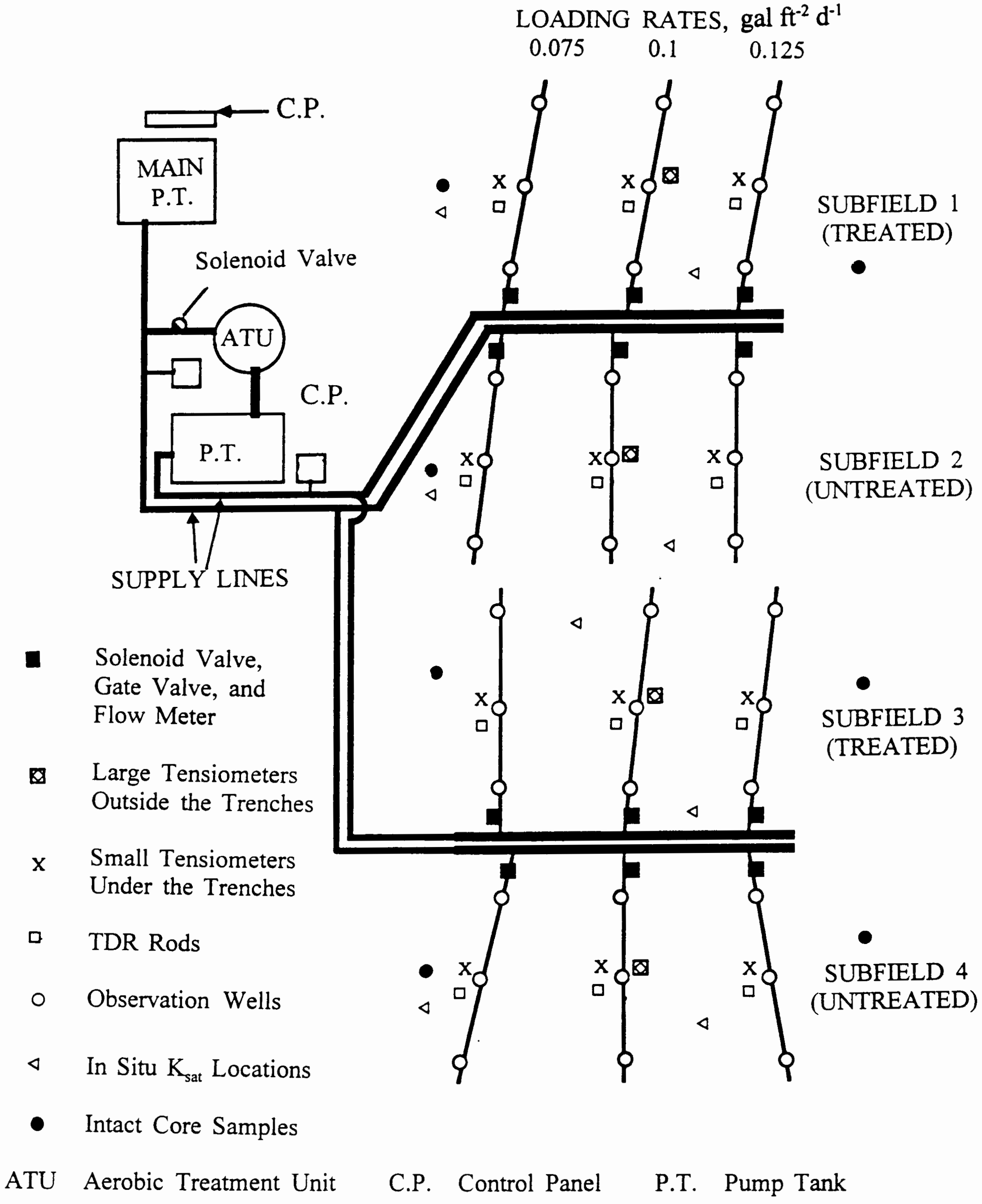

Figure 2. Schematic diagram of the plan view of the low-pressure pipe systems at the Wake County site showing the relative locations of the lateral lines and monitoring devices.

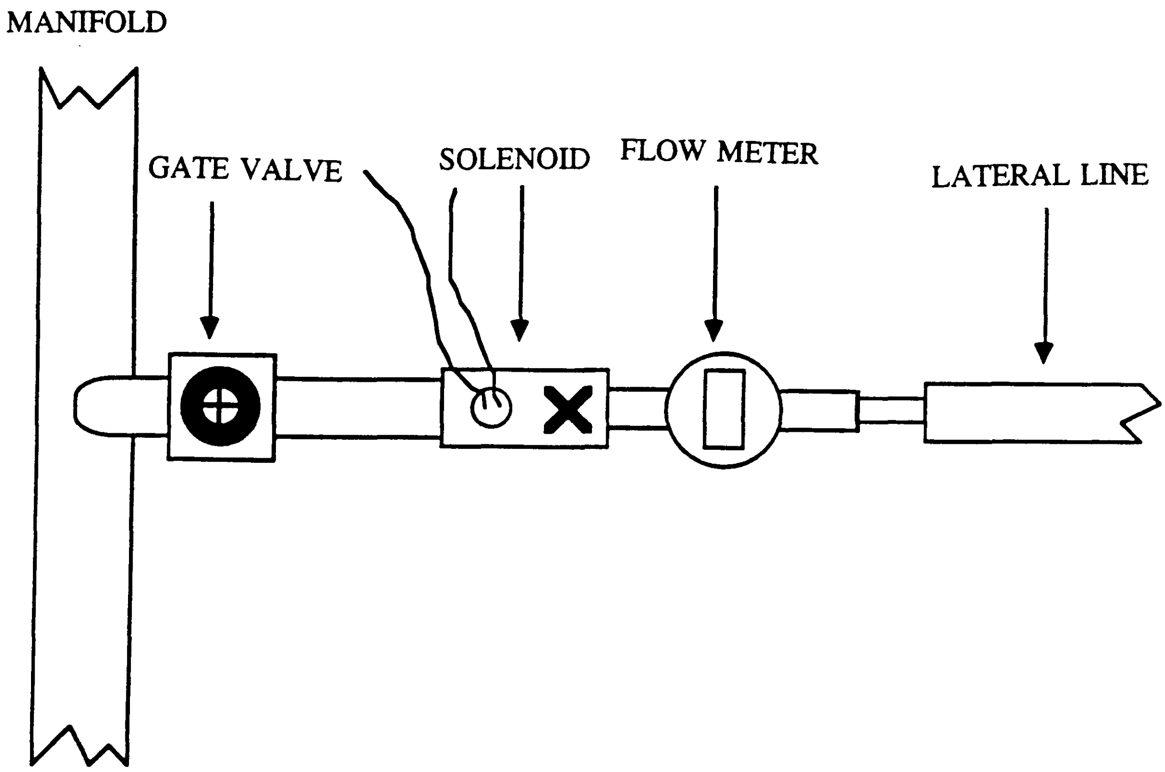

Figure 3. Schematic diagram of the gate valve, solenoid, and flow meter installed on each lateral line near the manifold of each low-pressure pipe system at the Wake County site.

Drip Systems

A drip system specifically designed for wastewater disposal (WasteWater Systems, Inc., Lilburn, Georgia) was used for the study at this site. This drip system is equipped with a relatively sophisticated electronic panel for controlling a pump and filter system to deliver filtered wastewater to the drainfield and to backwash the filter and lateral lines on a regular basis. The orifices on the drip lines are spaced 2-ft apart, and the minimum spacing between the lateral lines is 2 ft, as recommended by the manufacturer of the system (Waste Water Systems, Inc., 1992). For each of the two units we obtained for our study, the control system could be programmed for wastewater application (dosing) to two different subfields at different loading rates. For filtering wastewater before application to the drainfield, a 115-µm filter was used to remove suspended solids from wastewater drawn from the main pump tank of the system or the pump tank after the ATU. In our study, both treated and untreated wastewaters were applied to their respective drainfields at loading rates of 0.1 and 0.2 gal ft^{-2} d^{-1} (4 and 8 L m^{-2} d^{-1}, respectively) using two identical control-delivery systems.

To install the drip lines for each subfield, three 21-m (70-ft) long parallel trench lines that were 2 ft (60 cm) apart were marked in the designated area in the buffer zone of the main septic system. The manufacturer of this drip system assisted in installing the drainfields and connecting the control-delivery systems to the drainfields. A trencher, equipped for digging 4-in (10-cm) wide trenches, was used to construct trenches on the marked lines. To preserve the natural esthetics of the site under a natural forest, the spacing between parallel trenches that were installed deviated slightly from the 60-cm spacing in some areas. The trenches for the drip lines were 10 cm (4 in) wide and 20 to 40 cm (8 to 16 in) deep. The three trenches for each subfield were connected in series to form a 63-m (210-ft) long drip line. Figure 4 presents a schematic diagram of the areal view of the drip systems showing the relative locations of various subfields and the accompanying control-delivery systems.

The experimental drainfield area for the drip systems was in a relatively flat part of the landscape. Although we attempted to place the trenches for the drip lines on straight lines, no attempt was made to install the trenches on contour. In addition, the bottoms of the trenches were neither cleaned nor surveyed to assure their levelness. For each subfield, a continuous section of drip line was placed and stretched inside the three parallel trenches connected in series. Although the drip lines were pushed into the trenches, they were not forced to lay at the bottom of the trenches. In addition, no attempt was made to face the orifices in any specific direction. The technique used for installing the experimental systems (not requiring drainlines to be on contour, not requiring cleaning and leveling the bottoms of the trenches, and not requiring drip lines to have a specific orientation) was based on the best available guidelines at the time of installation.

The distance between the adjoining subfields for the two loading rates for each of the treated and untreated wastewater was approximately 120 cm (4 ft). The distance between the subfields for treated and untreated effluent was over 120 cm (4 ft). The design loading rate for each orifice on the drip lines was 0.6 gal h^{-1} (Waste Water Systems, Inc., 1992). The two loading rates of 0.1 and 0.2 gal ft^{-2} d^{-1} were obtained by dosing the respective subfields six times for approximately 7 and 14 min, respectively. Based on the areas of each subfield (70 ft by 6 ft), the daily wastewater applications for 0.1 and 0.2 gal ft^{-2} d^{-1} were 42 and 84 gal, respectively.

Soil and Site Characterization

Profile Description: Within the subfields of each of the LPP and the drip systems, the soil profile was described to approximately 2-m depth using the hand-auger boring technique and feel texture analysis. In addition to determining the texture of various horizons by the feel method, soil samples were collected from various depths/locations within the area for particle size distribution analysis in the laboratory. The pipet method (Gee and Bauder, 1986) was used for measuring sand, silt, and clay fractions of the samples.

Saturated Hydraulic Conductivity: The in situ saturated hydraulic conductivity (K_{sat}) of various depths/horizons was determined at 12 locations within the subfields of the LPP and drip systems by the constant-head well permeameter method (Amoozegar and Warrick, 1986). The relative locations of the measurements are shown in Figs. 2 and 4. At each location, a 6-cm diameter

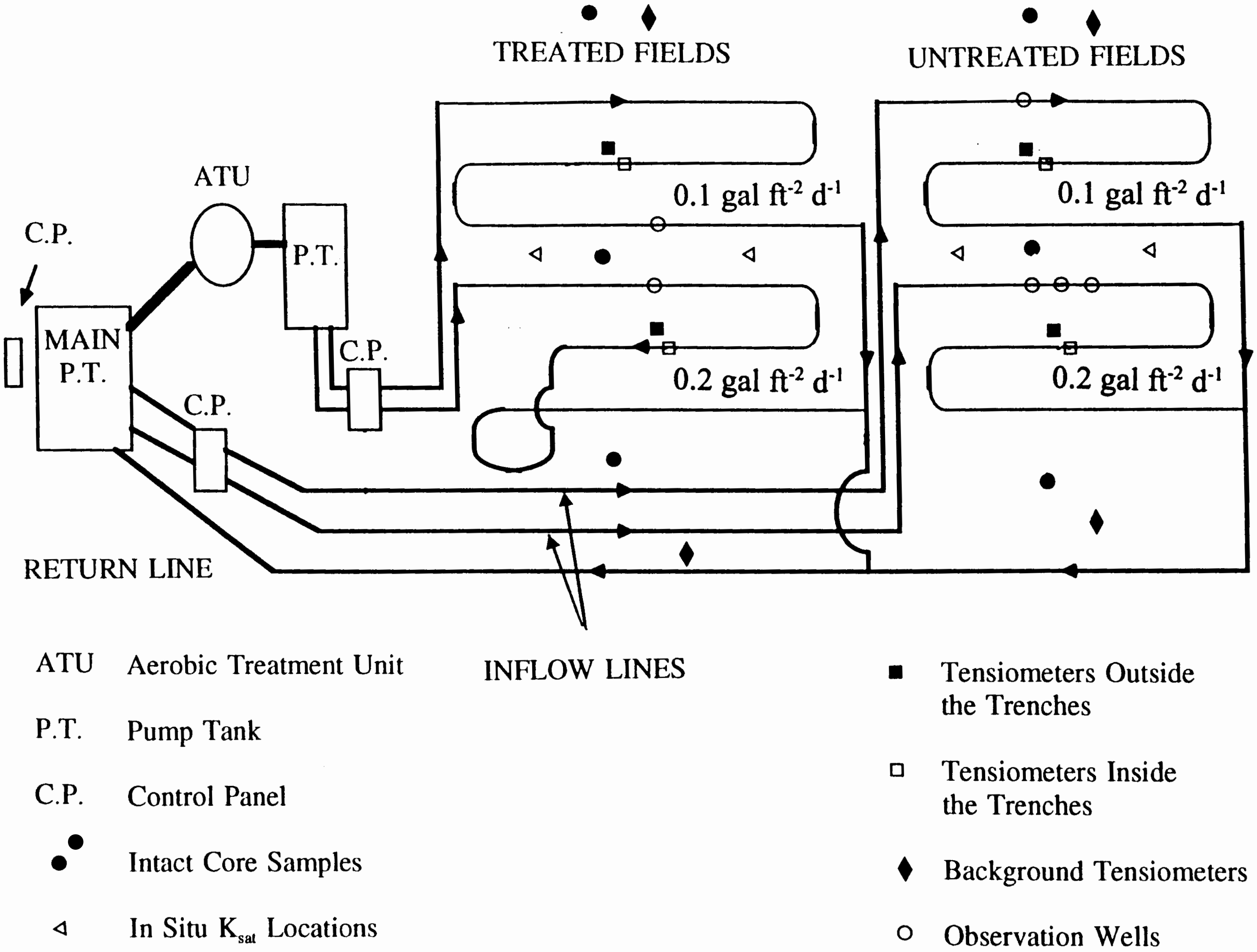

Figure 4. Schematic diagram of the plan view of the experimental drip systems showing the relative locations of the components and lateral lines for the treated and untreated wastewater. Also shown are the relative locations of the monitoring devices inside and outside the drip fields.

auger hole was bored to 25 cm depth, and the bottom of the hole was cleaned using a planer auger. The Compact Constant Head Permeameter (Amoozegar, 1989a) was used to maintain a 15-cm constant depth of water at the bottom of the auger hole. When steady-state was reached, the rate of water flow under the constant head was measured and K_{sat} was calculated by the Glover solution (Amoozegar, 1989b). Seven or more days were then allowed for the soil water around the auger hole to reach equilibrium with the surrounding soil, and the hole was then deepened to 50 cm depth. Saturated hydraulic conductivity was then measured and the procedure was repeated to measure K_{sat} at 25-cm depth intervals to 150 cm depth (25, 50, 75, 100, 125 and 150 cm depths) in the LPP subfields and to 125 cm depth (25, 50, 75, 100 and 125 cm depths) in the drip subfields.

One-hundred intact core samples were collected from various depths and locations within and around the drainfields of the LPP and drip systems as shown in Figs. 2 and 4, respectively. The intact samples (6.5 cm in diameter) were collected by a hydraulic probe, trimmed to 8 to 10 cm length, and prepared in the laboratory by the paraffin method described by Amoozegar (1988). Saturated hydraulic conductivity was determined in the laboratory by the constant head method (Klute and Dirksen, 1986).

Soil Water Characteristic: At the termination of K_{sat} measurements, 66 of the cores that were 10 cm long were cut into two intact pieces (8 and 2 cm long) for soil water retention analysis. All 8-cm long sections were used to measure soil water retention under 2.5, 5, 10, 15, 20, 30, and 40 kPa pressure (equivalent to -25, -50, -100, -150, -200, -300, and -400 cm pressure head, respectively). The 2-cm long intact sections were analyzed for soil water content at 100 kPa pressure (equivalent to -1,000 cm pressure head). Each of the intact 8-cm long cores was placed in a Buchner funnel, resaturated slowly from the bottom, and then subjected to various pressures to achieve the above soil water pressure heads (Klute, 1986). At the termination of this analysis a bulk density value was also obtained for each 8-cm long core. The short (2-cm long) section of each core was analyzed in a pressure plate apparatus as described by Klute (1986). Disturbed soil samples obtained from the above cores were used for measuring soil water content under 500 and 1,500 kPa pressure (equivalent to -5,000 and -15,000 cm pressure head, respectively).

Soil Water Pressure Head Measurement

The performance of each of the LPP and drip systems was evaluated by monitoring the soil water pressure head in and around their drainfield areas on a weekly basis. Two different sizes of tensiometers (1 cm in diameter, 2.4 cm long cup and 2.2 cm in diameter, 5 cm long cup, see West, 1994) and a commercially available portable pressure transducer described by Martheler et al. (1983) were used for this purpose.

For the LPP systems, three small tensiometers (referred to as pencil-size tensiometers) were installed through the access port in the middle part of each trench at vertical distances of 10, 30, and 50 cm below the bottom of the trench. The relative locations of these tensiometers are shown in Fig. 2. Due to the presence of rock fragments no tensiometer was installed at the 50 cm depth at two locations, and at another location a tensiometer was installed at the 40 cm instead of the 50 cm depth. The procedure for installation of the pencil-size tensiometers is described by West (1994).

A bank of large tensiometers containing five tensiometers at 50, 75, 100, 125, and 150 cm depths was installed on a line parallel to and 15 cm away from the middle trench wall in the middle of each subfield. To assure good contact between the cup and the surrounding soil, a hole slightly smaller than the diameter of the porous cup of the tensiometer was created by pushing a solid rod with a rounded tip into the soil. After extracting the rod, the tensiometer was inserted into the hole for a good contact.

For the drip systems, pencil-size tensiometers were used both inside and outside the trenches. A set of 12 tensiometers was installed on a 3-dimensional grid pattern around a section of the middle drip line in each of the subfields. For relative locations of these tensiometers see Fig. 4. For the grid pattern, tensiometers were installed on the vertices of three separate squares perpendicular to the drip line. For the middle square, one tensiometer was installed next to one orifice of the drip line and the other three tensiometers were installed at vertical and horizontal distances of 10 cm below and away from the drip line (Fig. 5A). The other two squares were located with the same arrangement at a 20-cm distance on each side of the middle square (Fig. 5B). Additional banks of tensiometers, each containing three tensiometers at depths of 25, 50, and 75 cm below the soil surface, were installed inside and outside the four subfields as shown in Fig. 4.

Soil Water Content Measurement

The volumetric soil water content from the soil surface to 60 cm depth in the LPP systems was measured weekly by the time domain reflectometry (TDR) technique (Dalton, 1992). Four pairs of TDR rods (15, 30, 45, and 60 cm long) were installed on a line parallel to and 15 cm from the trench wall in the middle of each trench (see Fig. 2). Using a Trase System (Soilmoisure Equipment Corp., Santa Barbara, CA), volumetric soil water content was measured for the entire length of each pair of rods (i.e., from the soil surface to 15, 30, 45, and 60 cm depths). These data were then converted to obtain soil water content for 0 to 15, 15 to 30, 30 to 45, and 45 to 60 cm depth intervals using weighted average soil water content for two successive depth intervals. The general equation for calculating soil water content (θ) for a given depth interval is

$$\theta_{i,j} = (j\theta_{0,j} - i\theta_{0,i})/(j-i),$$

where i and j ($i < j$) are two neighboring depths (e.g., 15 and 30 cm, respectively) associated with the lengths of the rods, and $\theta_{0,j}$ and $\theta_{0,i}$ are the soil water content values from the soil surface to j and i, respectively.

Measurement of Trench Ponding

Liquid levels in the beginning, middle, and end of each trench of the LPP systems were measured in a series of observation wells prior to and after wastewater application to the LPP systems on a weekly basis. A 60-cm long section of 2-inch aluminum irrigation pipe, containing

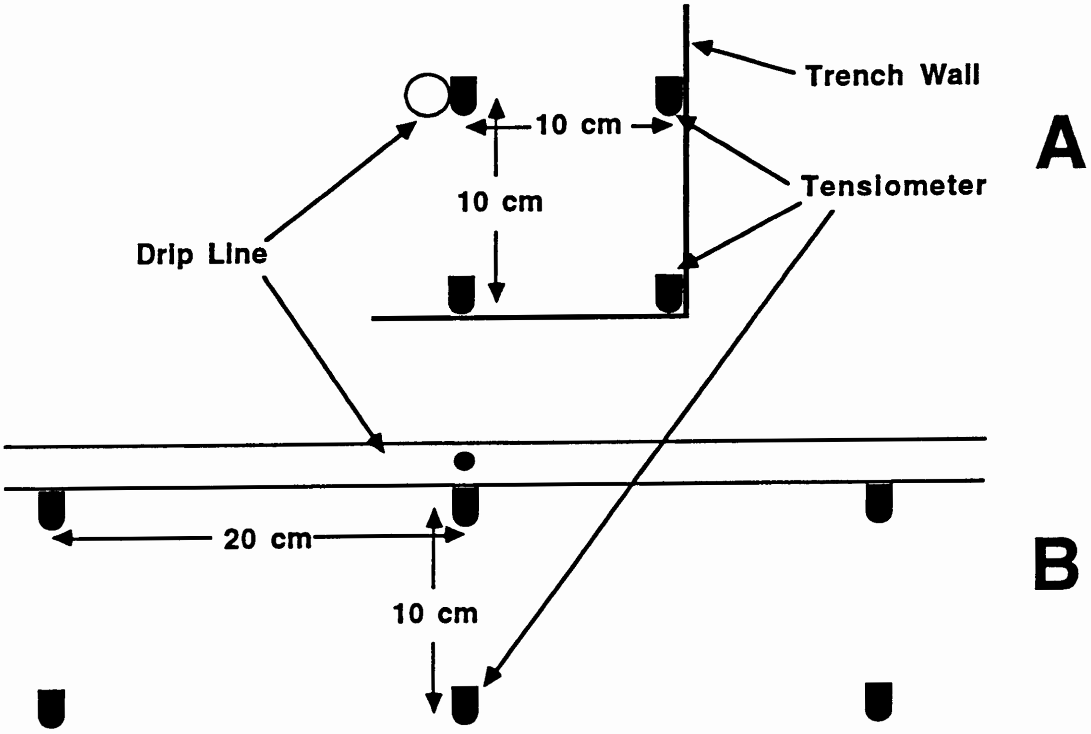

Figure 5. Schematic diagram of (A) the side view, and (B) the front view for the arrangement of pencil-size tensiometers installed around one emitter.

1-mm wide slits that were cut in its lower 10 cm were used as observation wells. The observation wells were located 1.2, 3, and 5.4 m from the manifold before the trenches were filled with gravel during installation of the LPP systems. The relative locations of the observation wells are shown in Fig. 2. To measure the wastewater level in each observation well, a wooden dowel was tightly attached to an empty 60-mL plastic bottle using a rubber stopper. The bottle was then placed inside the observation well and forced to the bottom of the trench. After marking a reference level on the dowel, the plastic bottle was allowed to float and the amount of rise of the marker above the reference level was measured as the depth of ponding in the trench. This procedure has been described by Martin (1987) and Weymann (1989). Wastewater level in each observation well was measured before a dosing cycle to determine the amount of ponding before wastewater application. Wastewater level was also measured immediately after the dosing event, and at 15 and 30 min after the dosing cycle was completed.

For the drip system, the same type of observation well and float system was used to monitor wastewater ponding in the trenches of the systems on a weekly basis. Only one observation well was placed in each subfield, except for the subfield receiving untreated effluent at 0.2 gal ft^{-2} d^{-1} where two additional observation wells were installed on both sides of the original well to monitor localized ponding (see Fig. 4). Because each drip system was dosed six times a day, wastewater level was not measured immediately before and at different time intervals after a dosing event. Instead, the wastewater level in each well was measured consistently at the same time of the day during the weekly monitoring of the system.

Wastewater Analyses

Wastewater samples were collected from different locations in the systems and analyzed for biochemical oxygen demand (BOD_5), chemical oxygen demand (COD), and fecal coliform bacteria. Samples were collected from the end of one lateral line of each of the LPP systems and from the lines of the drip systems receiving treated effluent and untreated wastewater. In the type of drip system used at the site, wastewater was filtered before being applied to the drip lines, and the filter was back washed prior to each dosing. Because of this filtering system, the manufacturer of this drip system recommended no further pretreatment of wastewater beyond the primary settlement of solids in the septic tank. The samples from the drip systems were taken to assess the capability of the filters in the delivery line of the drip system to filter solids and bacteria. The samples that were collected from various stages of the systems were submitted to a commercial laboratory for analysis. The membrane filter procedure (Greenberg et al., 1992) was used for the fecal coliform analysis. Analysis for fecal coliform for some of the samples were conducted in house using the same procedure.

Chatham County Site

This site was an existing septic system serving a single family dwelling. In general, the soils in the general vicinity of the research site in Chatham County are not suitable for subsurface septic systems under the present rules. Before the initiation of this study, a septic system composed of a septic tank, an ATU, a chlorinator, a storage (pump) tank, and a spray irrigation field had been installed by an independent contractor to serve a three-bedroom home on a wooded lot. The spray irrigation field contained five spray nozzles designed to cover an 810 m^2 (9000 ft^2) area with a design loading rate of 0.04 gal ft^{-2} d^{-1} or 1.14 cm wk^{-1} (0.45 in wk^{-1}). In this system, wastewater from the septic tank moved by gravity into the ATU where it received treatment. From the ATU, wastewater traveled through a chlorinator into a storage tank from which treated effluent was intermittently pumped into the spray field using a float system. Figure 6 presents a schematic diagram of the areal view of this system showing the relative locations of the septic tank, ATU, chlorinator, pump tank, and the disposal fields.

In addition to the existing spray field, a small subsurface drip system was installed above a drainage way near the spray irrigation field (Fig. 6). The drip system used at this site was a different brand (Geoflow Inc., Sausalito, CA) than the one used at the Wake County site. A distributor of the drip system assisted us in installing the system. Five 15-m (50-ft) long parallel trenches were dug at 60-cm (2-ft) spacing by the distributor of the drip system using a trenching machine. The distributor of the system did not require the trenches to be on contour, but the lines were dug perpendicular to the slope of the land and were nearly flat. The width of the trenches was 15 cm (6 in), and their depths ranged from 30 to 45 cm (12 to 18 in), with the shallowest trenches being the lower two trenches. Two trenches were also dug to connect the ends of the drip lines, and one of the trenches was extended to the pump tank. Drip lines were

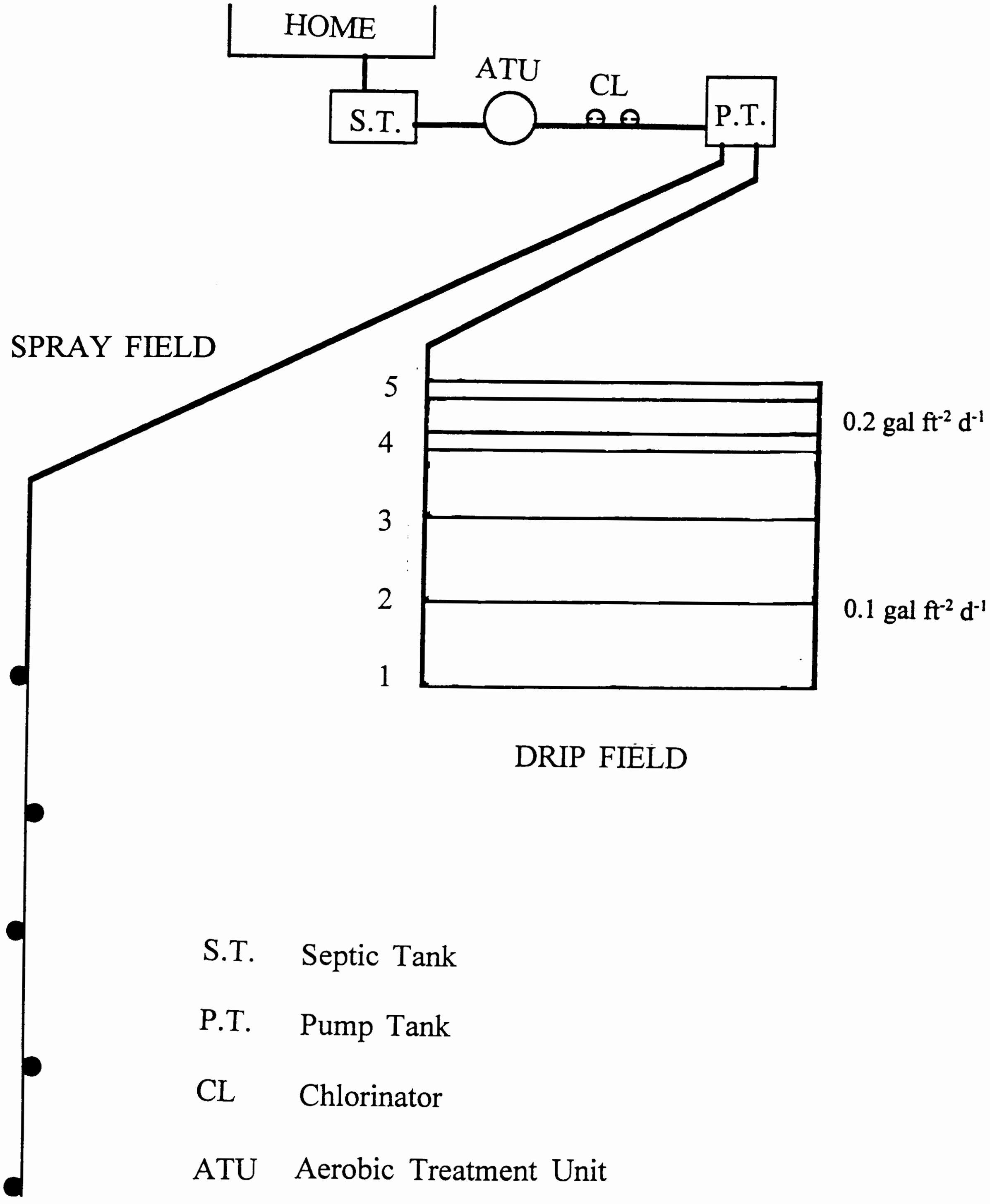

Figure 6. Schematic diagram of the plan view of the septic system at the Chatham County site showing the relative locations of the spray field and the drip system.

installed in each of the trenches and connected to a manifold at each end. Figure 7 presents the schematic diagram of the areal view of the drip system. One additional drip line was installed in each of the lower two trenches (lines 4 and 5) and they were connected to the other lines through a gate valve to allow doubling the loading rate in the lower two lines. A separate pump was installed in the pump tank of the spray irrigation system and a simple timer was used to apply wastewater to the drip system once per day. The loading rate was 0.05 gal ft^{-2} d^{-1} for lines 1, 2 and 3, and was 0.1 gal ft^{-2} d^{-1} for lines 4 and 5 (1 and 2 L m^{-2} d^{-1}, respectively). A flow meter was installed on the supply line to the drip field to monitor the dosing rate.

Soil and Site Characterization

Profile Description: The soil profile was described to 150 cm depth at three locations using hand-auger boring technique. Two of the locations were in the upper and lower parts of the drip field area and the third location was in the middle of the spray irrigation field. Soil samples were also collected and analyzed for particle size distribution by the pipet method (Gee and Bauder, 1986).

Saturated Hydraulic Conductivity: Intact core samples were collected from two locations outside the drip field (see Fig. 7) and from one location near the spray field. The maximum depth of samples for the two locations near the drip field was 115 cm, and the maximum depth near the spray field was 210 cm. Due to the restrictive nature of the subsoil in the area, intact samples could not be obtained by the hydraulic probe at deeper depths near the drip field. A total of 18 intact soil core samples could be obtained from the three locations. These samples were prepared in the laboratory and analyzed for K_{sat} by the constant head method as described earlier.

In situ K_{sat} of four depth intervals was measured at three locations within the drip field and three locations within the spray field. The depth intervals corresponded with the combined A and E, B and C horizons as determined during the boring of the auger holes for measuring K_{sat}. The maximum depth of in situ measurements was 100 cm within the drip field and 85 cm within the spray field.

Soil Water Characteristics: After measuring K_{sat} of the intact cores in the laboratory, the cores were used to determine soil water content under 0, 2.5, 5, 10, 20, 30, and 40 kPa pressure (equivalent to 0, -25, -50, -100, -200, -300, and -400 cm of pressure head, respectively) as described for the Wake County site. In addition, soil water contents of 18 intact cores were measured under 100 kPa pressure (equivalent to -1,000 cm of pressure head). Soil water contents at 500 and 1,500 kPa pressure (equivalent to -5,000 and -15,000 cm of pressure head, respectively) were measured using disturbed samples. As part of the soil water characterization, the bulk density of each sample was also measured.

Spray Irrigation Distribution

The uniformity of wastewater distribution over the spray field was assessed by collecting wastewater at a number of predetermined locations inside the spray field (Fig. 8) at different times of the year. Catch cans, 15 cm in diameter, were placed at the predetermined locations to

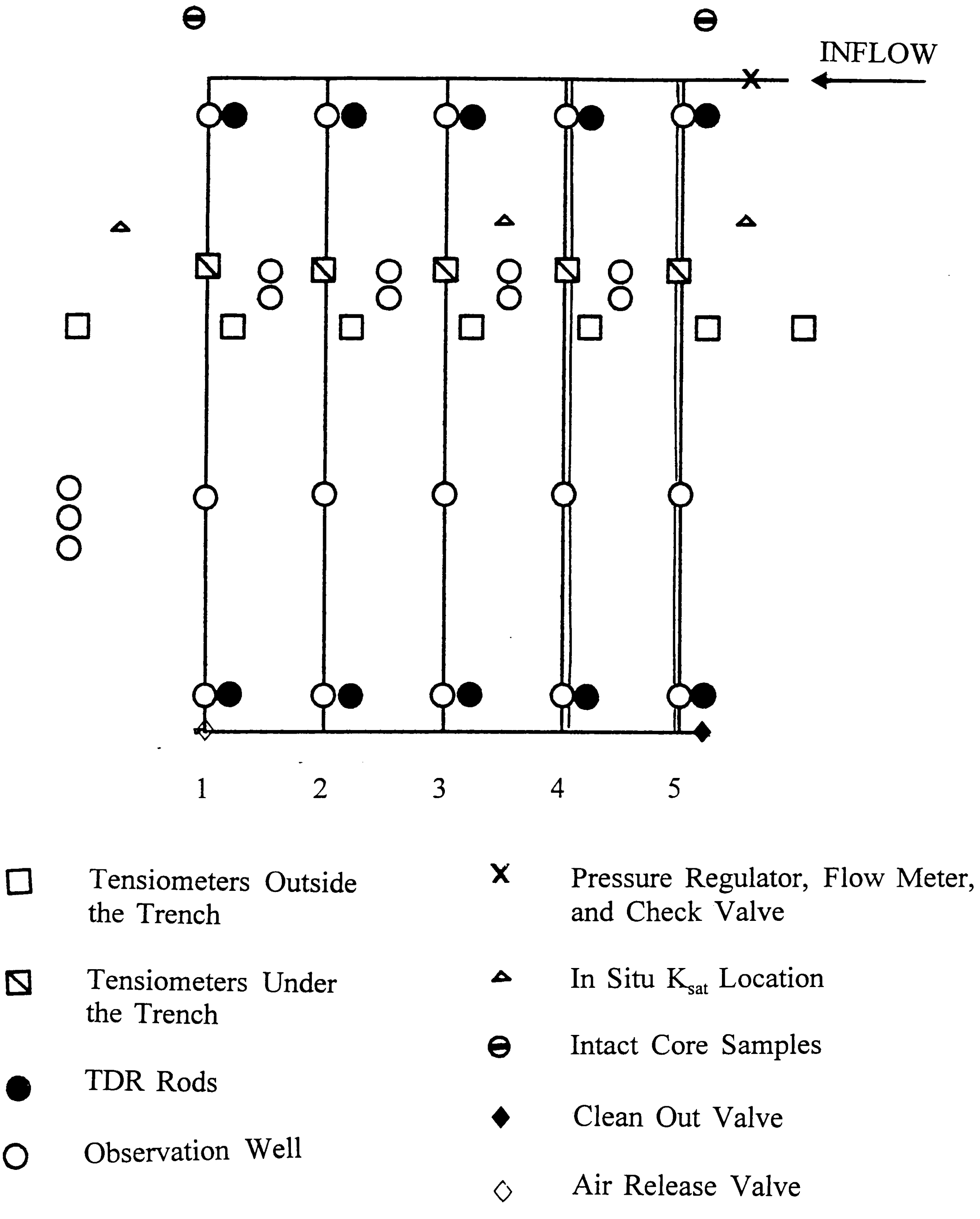

Figure 7. Schematic diagram of the plan view of the drip system at the Chatham County site showing the relative locations of the monitoring devices and sample locations.

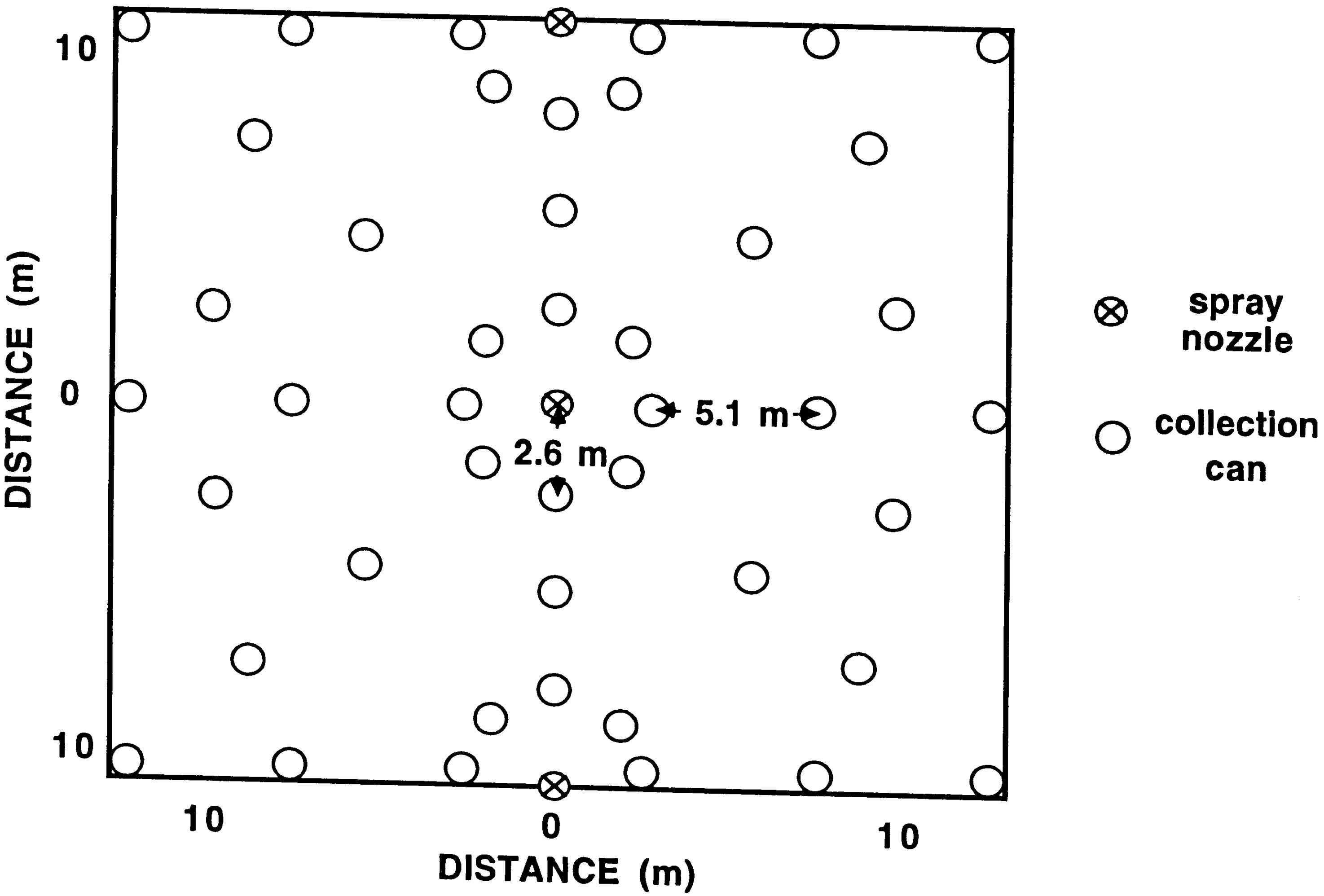

Figure 8. Schematic diagram of the areal view of part of the spray irrigation system showing the relative locations of the collection cans for determining the uniformity of wastewater distribution over the spray field.

cover the impacted area of three adjoining spray nozzles. This area included the full circle of the central nozzle and the two half-circles of the adjoining nozzles. As shown in Fig. 8, the spacing between the catch cans were approximately 2.6 or 5.1 m. The 44 locations for this set up were selected to cover the impacted area. At one location, near a tree, an additional catch can was employed to catch splattering from the tree. Measurements were conducted during the winter and early spring when foliage and underbrush vegetation were not a major factor, and during the summer when vegetation was at its peak.

Initially, the cans were placed at their locations and the amount of liquid in each can was measured weekly. This gave the overall distribution of wastewater and precipitation reaching the ground over the spray field. To eliminate the potential error resulting from nonuniformity of rainfall distribution and evaporation from the cans, the spray irrigation distribution was assessed for individual dosing events rather than weekly monitoring. For this analysis, empty cans were placed in their respective positions, and the system was activated (dosed) for approximately 30 minutes. Sufficient time was then allowed for dripping from plants to cease and for the spray field to become relatively dry before entering the field to measure the volume of wastewater in each can for that spray event. For all measurements, the data were then converted from volume into the depth of wastewater over a unit area and interpolated by a finite difference scheme to prepare plots of wastewater distribution over the measurement area.

Soil Water Pressure Head Measurement

Pencil-size tensiometers were installed at 0 and 20 cm below the bottom of the trench at one location within each lateral line (see Fig. 7). Regular tensiometers were installed at three depths (20, 40, and 60 cm) outside of each trench. The pencil-size tensiometers were 6 m from the manifold and the regular size tensiometers were located 7.5 m from the manifold. In addition to the tensiometer banks inside the drip field, two banks of regular size tensiometers were installed above and below the drip field. Soil water pressure head was measured weekly using the portable pressure transducer that was mentioned earlier.

Soil Water Content Measurement

Soil water content at two locations around each of the drip lateral lines was measured by TDR. Banks of TDR rods, each containing four pairs of TDR rods (15, 30, 45, and 60 cm long), were installed to measure the volumetric soil water content from the soil surface to 15, 30, 45, and 60 cm depth intervals. The banks of TDR rods were installed at a distance of 15 cm on a line parallel to each drip line and at a distance of 60 to 90 cm from the manifolds at the ends of each line. The relative locations of the TDR rods are shown in Fig. 7. Soil water content was measured weekly using a Trase System. As explained earlier, the data were then converted to calculate soil water content at 0 to 15, 15 to 30, 30 to 45, and 45 to 60 cm depth intervals.

Measurement of Trench Ponding for the Drip System

Three observation wells (60-cm long section of 2-in aluminum pipe with 1-mm slots at one end) were installed at the beginning (60 cm from the front manifold), middle (9 m from the front manifold), and end (14 m from the front manifold) of each drip lateral line during the installation of the drip system. Two observation wells with 10 cm long perforations at the bottom were also installed in the A and/or E and in the B horizon half-way between the lateral lines. For relative locations of observation wells refer to Fig. 7. To monitor the possible influence of the spray field and the lateral flow into the drip field, an additional bank of observation wells with three wells was installed between the drip field and spray field to monitor the A and E; the A, E and B; and exclusively the B horizons. A similar type of float device used for the Wake County site was used to measure the level of liquid in each observation well.

Wastewater and Runoff Analyses

Wastewater samples were collected from the septic tank (before ATU), the ATU (before chlorination), the pump tank (after chlorination), and from one observation well located within one of the drip lines that was continuously ponded. These samples were analyzed for BOD_5, COD, and fecal coliform by an independent laboratory as described for the Wake County site.

Runoff samples were periodically collected from the spray field using a wooden flume (William O. Thompson, Ag. Res. Tech II, NCSU, personal communication, 1993). The samples were analyzed for electrical conductivity (EC), pH, $N\text{-}NO_3$, and $N\text{-}NH_4$. The $N\text{-}NO_3$ and $N\text{-}NH_4$ were analyzed by the automated cadmium reduction method and automated phenate method, respectively (Greenberg et al., 1992).

Distribution Uniformity of Drip Systems

The fundamental differences between the two drip systems used in this study are the configuration and design of the drip orifices, the layout of the drip lines, and the mode of operation of the systems. The system used at the Wake County site uses drip lines (Netafim Irrigation, Inc., Valley Stream, New York) that have a pressure compensating orifice with one hole at every 2-ft interval. The system used at the Chatham County site, on the other hand, does not use a compensated pressure orifice mechanism in its drip lines, but has two holes on the opposite side of the line at 2-ft intervals. [NOTE: In this report, the pair of orifices at each location along the drip lines for the system used at the Chatham County site will be referred to as an orifice.] Although the orifice configurations of the drip systems were different, both consisted of ½-in plastic tubing with orifices on 60-cm (2-ft) intervals.

Our field data for the drip systems indicated that wastewater was not distributed uniformly over their respective drainfield areas. To assess the uniformity of wastewater distribution by the drip systems, a series of measurements were performed using above ground drip systems and tap water. For this purpose, drip lines of both systems used in the field experiments were used.

Whether installing the drip lines in trenches and then backfilling with soil alone (i.e., without any gravel), or installing the lines in slits created in the soil by a vibratory plow, at the time of this study there was no specific requirement to maintain the drip lines on contour. We should note that due to the unevenness of the soil surface (particularly at forested sites), it is difficult to install the drip lines at zero slope by either a vibratory plow or a trencher. Also, the orifices on the lines could face any direction in actual installation. That is, no attempt is made to keep the orifices downward during field installation. For our above ground experiments, first we installed the drip lines on a level surface (i.e., zero slope along the line) with the orifices having a random orientation. To achieve a zero slope, a series of nails was installed along a straight line on the side of 2 by 4 studs. The studs were then mounted on blocks placed on a concrete slab to achieve zero slope for the line of nails. The drip lines were then laid on top of the nails, and the elevation of each line was surveyed. Figure 9 presents a schematic diagram of the layout for the two experiments. To avoid losing water from the orifices, a piece of paper towel was loosely wrapped around each orifice for collecting water in a small beaker.

Each drip system was composed of seven 13-m (42-ft) long drip lines (shown schematically in Fig. 9). Tap water was applied to the system used at the Wake County site under 50 psi (35 m of head) pressure and to the other system under 20 psi (14 m of head) pressure based on the recommended value for each system. For each system, a water hose attached to a water faucet was connected through a small ball valve to one end of the system, and a regulator with a gauge was used to adjust the pressure. Initially, water was applied to each system and allowed to flow out of all orifices for at least one hour before initiating the measurements. Measurement of flow from orifices was initiated after complete free drainage from the drip lines. Small beakers (500 mL) were used to collect water from 20 orifices at the beginning (line No. 1), 20 orifices in the middle (line No. 4), and 20 orifices at the end (line No. 7) of each system. Water from each of these orifices was collected and measured for a 3-min period immediately after starting water application to the lines, for a 3-min period in the middle of water application (when the system was fully pressurized), and for the last 3 min of water application followed by the free drainage period. [NOTE: There were up to 30 seconds difference in the duration of water collection from orifices on each line for the first and last measurements. The actual time for each orifice has been used for calculation.] For the last measurement, collection of water from each orifice was continued until drainage from the orifices was complete. The same experiment was repeated two more times, once by varying the elevation of orifices on the drip lines within a 5-cm range to simulate a hypothetical field installation at a flat site, and once by creating a uniform slope of 1.25% along the drip lines.

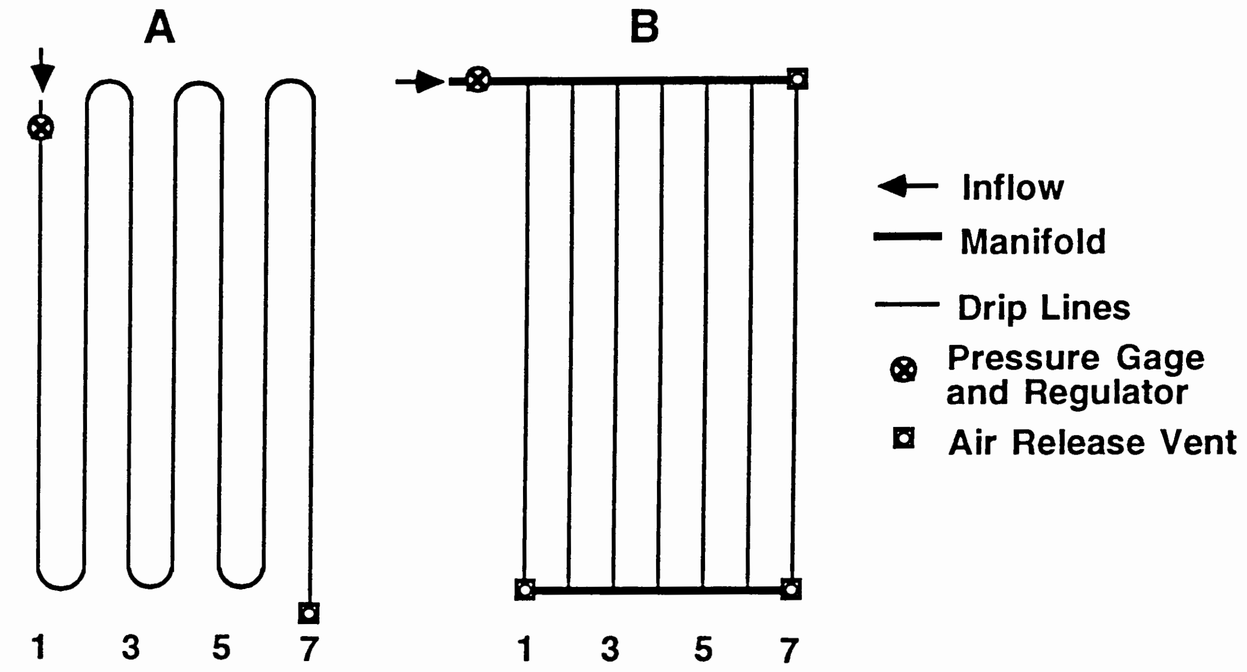

Figure. 9. Schematic diagram of the plan view of the above ground drip systems used at the (A) Wake County and (B) Chatham County sites showing the relative locations of the laterals, the manifolds, and air vents. Line numbers are shown at the bottom of each system.

RESULTS AND DISCUSSION

For all results expressed with time as a variable (x-axis in figures labeled as "DAYS"), day 1 refers to January 1, 1993, as the start of the monitoring period.

Wake County Site

Soil and Site Characterization

Profile Description: As indicated earlier, two different types of soils were identified over the LPP drainfields during the installation of the four subfields, and the soil profiles at all four subfields were described in the field using hand-auger boring technique. In the upper part of the LPP area (subfield 1, see Fig. 2), the A and BA horizons together were 30 cm thick, and the Bt horizon (composed of Bt1 and Bt2), on the average, was found between 30 and 127 cm depth. The transitional BC horizon was from 127 to 155 cm depth. The soils in the upper part of the LPP drainfields were identified as Appling (a clayey, kaolinitic, thermic Typic Kanhapludult). Moving from subfield 1 toward subfield 4, the A horizon was thinner (5 to 8 cm thick), and the Bt horizon was no more than 55 cm deep. In subfields 3 and 4, saprolite was identified at 90 cm and 70 cm depths, respectively. The soils at the lower part of the subfields were identified as Pacolet (a clayey, kaolinitic, thermic Typic Kanhapludult). The profile descriptions for all subfields are presented by West (1994).

Over the drip subfields, both Pacolet and Cecil (also a clayey, kaolinitic, thermic Typic Kanhapludult) soil series were identified (see West, 1994). In the area that received treated effluent, the A and Bt horizons were rather thin (8 and 38 cm thick, respectively), and the soils were very similar to the soils in LPP subfields 3 and 4. The soils in the subfields for treated effluent were placed in the Pacolet series. In this area of the site, saprolite was noticed at 70 cm below the soil surface. Over the area where untreated effluent was applied, the soil was classified as Cecil. The A horizon was 16 cm thick and the transitional Bt/C horizon was near the 100 cm depth. Saprolite over the area receiving untreated effluent was identified at 145 cm depth.

Particle Size Distribution: Table 1 presents the particle size distribution for the LPP and drip fields. For LPP subfields 1 and 2 the maximum clay content of the Bt horizon was 46.2%, whereas for subfields 3 and 4 the clay content was as high as 71.2%. Saprolite under subfield 1 was deeper than 150 cm, but for the other fields, the clay content was reduced substantially from 75 to 100 cm depth and reached a low value of 12.8% at 150 cm depth. The sand content data also indicated that saprolite started at a depth greater than 150 cm under subfield 1, and perhaps as shallow as 125 cm under subfield 4. For the drip systems, the maximum depth of sampling was 80 cm, and the maximum clay content of the Bt horizons was 66%. Because the drip lines were installed shallow, we did not see it necessary to analyze the soil texture beyond the 80 cm depth. Based on the textural data, it appears that the Bt and the transitional horizon (BC) were thicker than found in the profile description using auger borings. This indicates that auger boring technique is inadequate for saprolite identification, and that saprolite should be evaluated using observation pits as recommended by Amoozegar et al. (1993).

Table 1. Particle size distribution with depth for the four LPP and two drip subfields at the Wake County site.

		DEPTH, cm						
FIELD	PARTICLE SIZE	Surface	25	50	75	100	125	150
		%						
LPP #1	Sand	65.0	57.7	40.2	37.5	53.2	51.3	26.2
	Silt	24.5	23.2	19.0	16.3	11.0	18.7	32.5
	Clay	10.5	19.1	40.8	46.2	35.8	30.0	41.3
LPP #2	Sand	67.2	60.1	39.3	44.3	53.0	32.3	60.8
	Silt	23.6	17.1	14.5	13.4	14.0	33.1	24.7
	Clay	9.2	22.8	46.2	42.3	33.0	34.7	14.6
LPP #3	Sand	62.5	27.2	22.7	28.1	33.6	31.0	45.1
	Silt	21.5	11.5	10.9	10.7	35.3	41.0	39.7
	Clay	16.0	61.3	66.4	61.2	31.1	28.0	15.2
LPP #4	Sand	16.4	16.5	17.2	20.3	36.1	54.7	70.8
	Silt	67.3	12.3	12.4	15.5	26.2	25.3	16.3
	Clay	16.3	71.2	70.4	64.2	37.7	20.0	12.8

		DEPTH INTERVALS, cm							
		0-10	10-20	20-30	30-40	40-50	50-60	60-70	70-80
		%							
Treated Drip	Sand	67.2	47.9	33.2	29.2	34.4	36.6	41.5	43.8
	Silt	19.0	13.5	10.2	9.7	10.3	9.9	10.5	13.3
	Clay	13.8	38.6	56.6	61.1	55.3	53.5	48.0	42.9
Untreated Drip	Sand	61.0	19.4	20.6	19.6	17.9	18.1	28.1	20.1
	Silt	18.5	14.6	14.5	18.2	17.8	21.3	19.2	19.2
	Clay	20.5	66.0	64.9	62.2	64.3	60.6	52.7	60.7

Bulk Density: Overall the bulk density of core samples collected from subfield 1 was the highest with a range of 1.40 to 1.72 g cm^{-3}. The bulk density values for subfields 2, 3, and 4 ranged from 1.35 to 1.65, 1.08 to 1.54, and 1.24 to 1.62 g cm^{-3}, respectively. The gradual increase in the bulk density from subfield 4 to subfield 1 corresponds with higher sand content in the Bt horizons of subfields 1 and 2 as compared to subfields 3 and 4.

For the drip systems, the range of bulk density values for the treated field was 1.13 to 1.53 g cm^{-3} with a mean of 1.41 g cm^{-3}. For the deeper soil in the area receiving untreated wastewater (Cecil soil), the range of bulk density was 1.13 to 1.64 g cm^{-3}. These results also correspond well with the results for particle size distribution values that are reported in Table 1.

Saturated Hydraulic Conductivity: The in situ and laboratory-determined K_{sat} yielded similar trends for individual subfields for both LPP and drip systems (Figs. 10 and 11). In general, K_{sat} decreased with depth from the top of the Bt and reached a minimum value in the transitional (BC or B/C) horizon followed by a modest increase in the saprolite at each subfield. The trends observed at this site were similar to the ones reported by Simpson (1986), Schoeneberger et al. (1995), and Amoozegar et al. (1993) for different soils in the Piedmont region of North Carolina. Considering the variability of K_{sat} (Warrick and Nielsen, 1980), the potential of smearing the auger hole sidewall, or obtaining a core sample containing macropores (e.g., cracks, root channels), the overall agreement between the in situ and laboratory K_{sat} values is better than expected.

In the upper part of the LPP area (subfields 1 and 2 in Fig. 2), the minimum in situ K_{sat} value was for the BC horizon (0.1 cm d^{-1}) at a depth of approximately 100 cm (Fig. 10A). In the lower part of the LPP systems, the lowest K_{sat} values were 0.24 cm d^{-1} at 35 to 50 cm depth in subfield 4 and 0.34 cm d^{-1} at 60 to 75 cm depth in subfield 3, respectively (Fig. 10B). For the drip systems, the minimum measured K_{sat} value was 0.2 cm d^{-1} at the 35 to 50 cm depth interval (Fig. 11). A layer or horizon with a low K_{sat} at a distance below the bottom of the trenches in the LPP or the drip lines in the drip systems may restrict downward movement of wastewater through the profile and induce a saturated zone at or above the transitional (BC or B/C) horizons.

A major requirement for septic systems in North Carolina is the presence of at least 30 cm of unsaturated, suitable soil between the bottom of the trenches and any restrictive layer such as rock or ground water. When the soils below the trenches restrict downward water movement, a saturated zone may be formed below the trenches, and the required 30 cm separation distance may not be achievable. Although no surface failure may result from such an action, the treatment capability of the system will be compromised. In the Piedmont region, the occurrence of a relatively low permeable zone between the Bt and C (saprolite) horizons may create a saturated zone below the trenches of a LPP or drip lines of a drip system. In our study, the trenches for the LPP systems were 45 to 50 cm deep. Based on the profile description and measured K_{sat}, the trenches in subfields 1 and 2 were in the Bt and at least 50 cm above the most restrictive layer. In subfields 3 and 4, on the other hand, the trenches were on top of the low conductivity BC or B/C layers. In the subfields of the drip systems, the trenches for the drip lines were a considerable distance above the restrictive layer. Obviously, this may facilitate a

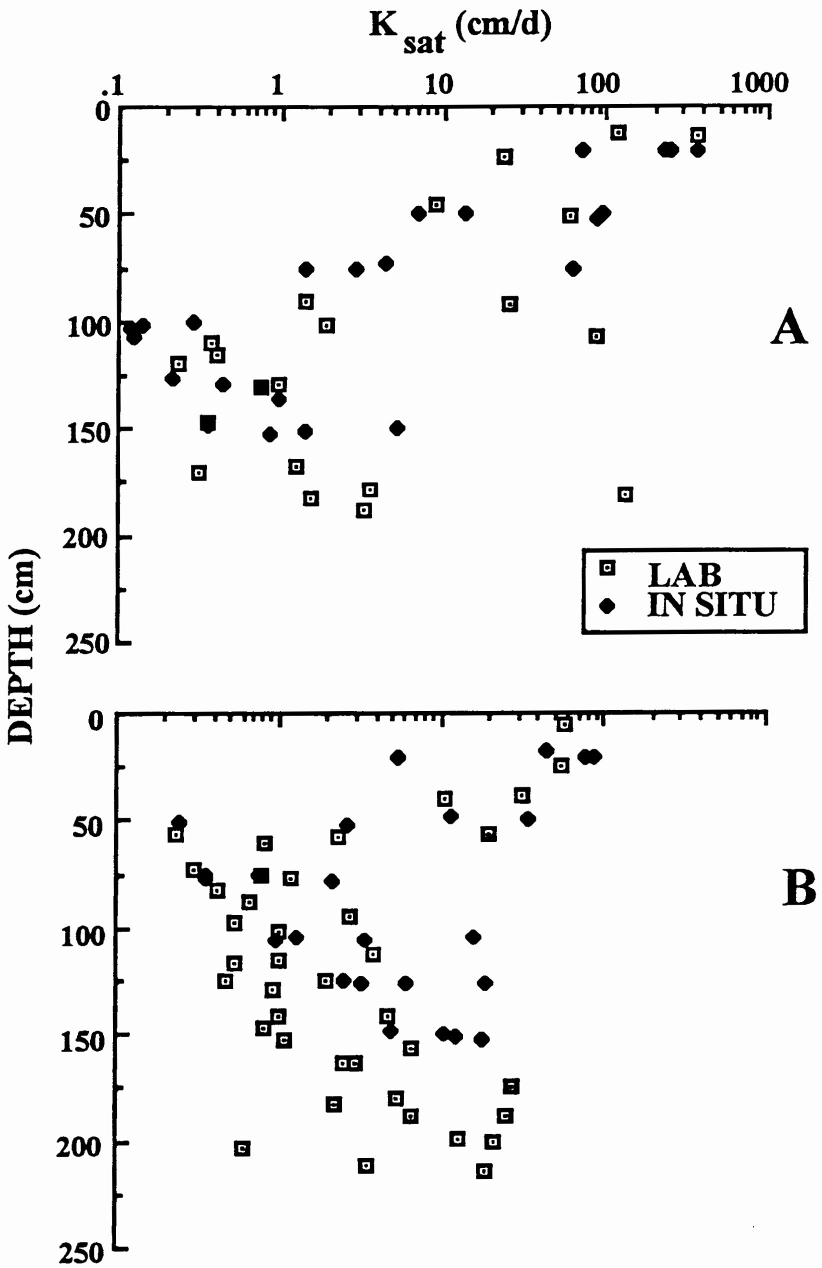

Figure 10. In situ and laboratory-determined K_{sat} for (A) subfields 1 and 2, and for (B) subfields 3 and 4 at the Wake County site.

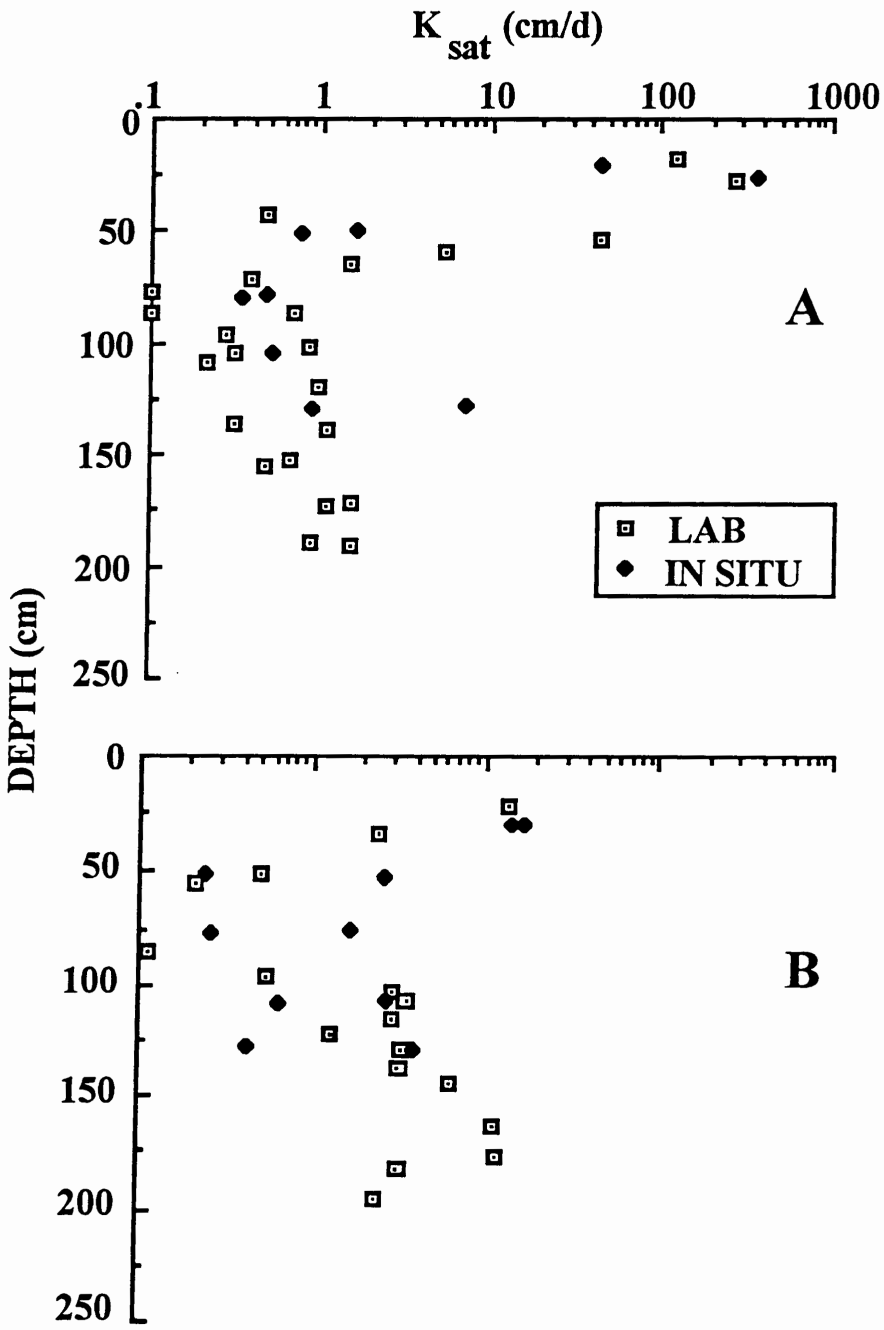

Figure 11. In situ and laboratory-determined K_{sat} for the drip systems for (A) untreated wastewater and for (B) treated wastewater at the Wake County site.

better distribution of wastewater under the systems, and may allow deep percolation without inducing saturated conditions directly beneath the trenches.

Soil Water Characteristic: The soil water retention under different pressure heads (soil water characteristic) for intact core samples obtained from the four subfields of the LPP and the treated and untreated areas of the drip systems are presented in Table 2. For each subfield, the value at each soil water pressure head is the average value for all cores collected from various horizons (depths). The standard deviation for each mean is also given in the table. For the drip systems, the average soil water content for each pressure head for the untreated field as reported in Table 2 is greater than the corresponding value for the treated field. For the LPP systems, on the other hand, all four values for each soil water pressure are close. In fact, the soil water pressure head values for the treated drip field are fairly near the corresponding values for the LPP subfields 3 and 4. Note that the soil in the untreated drip field was generally deeper and more clayey than the other soils in the treated drip and the LPP subfields.

Low-Pressure Pipe Systems

Ponding in the Trenches: We start with analyzing the ponding in the trenches of the LPP systems before and after dosing for five different dates (Figs. 12 to 15). For comparison, the weekly precipitations for the measurement dates are also given in the figures. The three main graphs in each figure are the ponding before and after a dosing for the three loading rates (0.075, 0.1, and 0.125 gal ft^{-2} d^{-1}). The positions 1, 2, and 3 correspond with the locations of the observation wells at 1.2, 3, and 5.4 m from the manifold in each trench, respectively.

For subfields 1 and 3 receiving treated effluent, ponding occurred very infrequently as shown in Figs. 12 and 13. For the five periods reported in Fig. 12, ponding occurred at a few locations only at one time that corresponded with high precipitation. Similar results were observed for subfield 3 (Fig. 13). Overall for treated subfields, almost no ponding was observed when precipitation was less than 1 cm wk^{-1}, very little ponding was measured when precipitation was between 1 and 1.5 cm wk^{-1}, and the maximum ponding occurred when weekly precipitation exceeded 1.5 cm. For subfield 2 receiving untreated effluent (Fig. 14), ponding occurred only when precipitation exceeded 1 cm wk^{-1}, while for subfield 4 (Fig. 15) ponding occurred frequently at various locations for all three loading rates regardless of precipitation. This is perhaps due to a thinner A and shallower depth to the low conductivity BC horizon. As indicated previously, the lowest measured K_{sat} (0.24 cm d^{-1}) within subfield 4 was at 50 cm depth, which is very near the bottom of the trenches. At no time during the study was wastewater observed at the soil surface. However, higher ponding levels in the trenches of the subfields receiving untreated wastewater compared to the ponding level in the trenches of the subfields receiving treated wastewater are the result of low K_{sat} around the bottom area of the trenches as well as the quality of the effluent.

Table 2. Mean, standard deviation (s.d.), and number of observations (n) for soil water retention under various soil water pressure heads for the four LPP and two drip fields at the Wake County site.

		SOIL WATER PRESSURE HEAD, cm										
FIELD		0	-25	-50	-100	-150	-200	-300	-400	-1,000	-5,000	-15,000
		$m^3 m^{-3}$										
LPP #1	Mean	0.477	0.463	0.451	0.433	0.421	0.413	0.402	1.393	0.373	0.264	0.219
	s.d.	0.075	0.076	0.080	0.086	0.086	0.086	0.085	0.085	0.085	0.083	0.076
	n	13	13	13	13	13	13	13	13	10	13	13
LPP #2	Mean	0.523	0.500	0.494	0.476	0.458	0.448	0.436	0.427	0.333	0.221	0.175
	s.d.	0.049	0.046	0.040	1.033	0.039	0.041	0.044	0.047	0.051	0.060	0.058
	n	9	9	9	9	9	9	9	9	4	9	9
LPP #3	Mean	0.528	0.513	0.508	0.493	0.475	0.463	0.444	0.429	0.404	0.274	0.217
	s.d.	0.046	0.047	0.043	0.044	0.048	0.053	0.058	0.063	0.057	0.079	0.076
	n	16	16	16	16	16	16	16	16	15	16	16
LPP #4	Mean	0.536	0.515	0.512	0.487	0.464	0.447	0.429	0.415	0.405	0.259	0.203
	s.d.	0.038	0.049	0.044	0.047	0.053	0.059	0.067	0.074	0.091	0.097	0.089
	n	22	22	22	22	22	22	22	22	13	22	22
Treated Field	Mean	0.531	0.513	0.509	0.488	0.469	0.458	0.446	0.437	0.363	0.272	0.219
	s.d.	0.038	0.041	0.038	0.044	0.053	0.057	0.03	0.067	0.037	0.102	0.091
	n	17	17	17	17	17	17	17	17	7	17	17
Untreated Field	Mean	0.550	0.530	0.528	0.515	0.508	0.502	0.494	0.487	0.535	0.387	0.316
	s.d.	0.061	0.073	0.070	0.072	0.073	0.073	0.074	0.073	0.036	0.068	0.063
	n	23	23	23	23	23	23	23	23	17	23	23

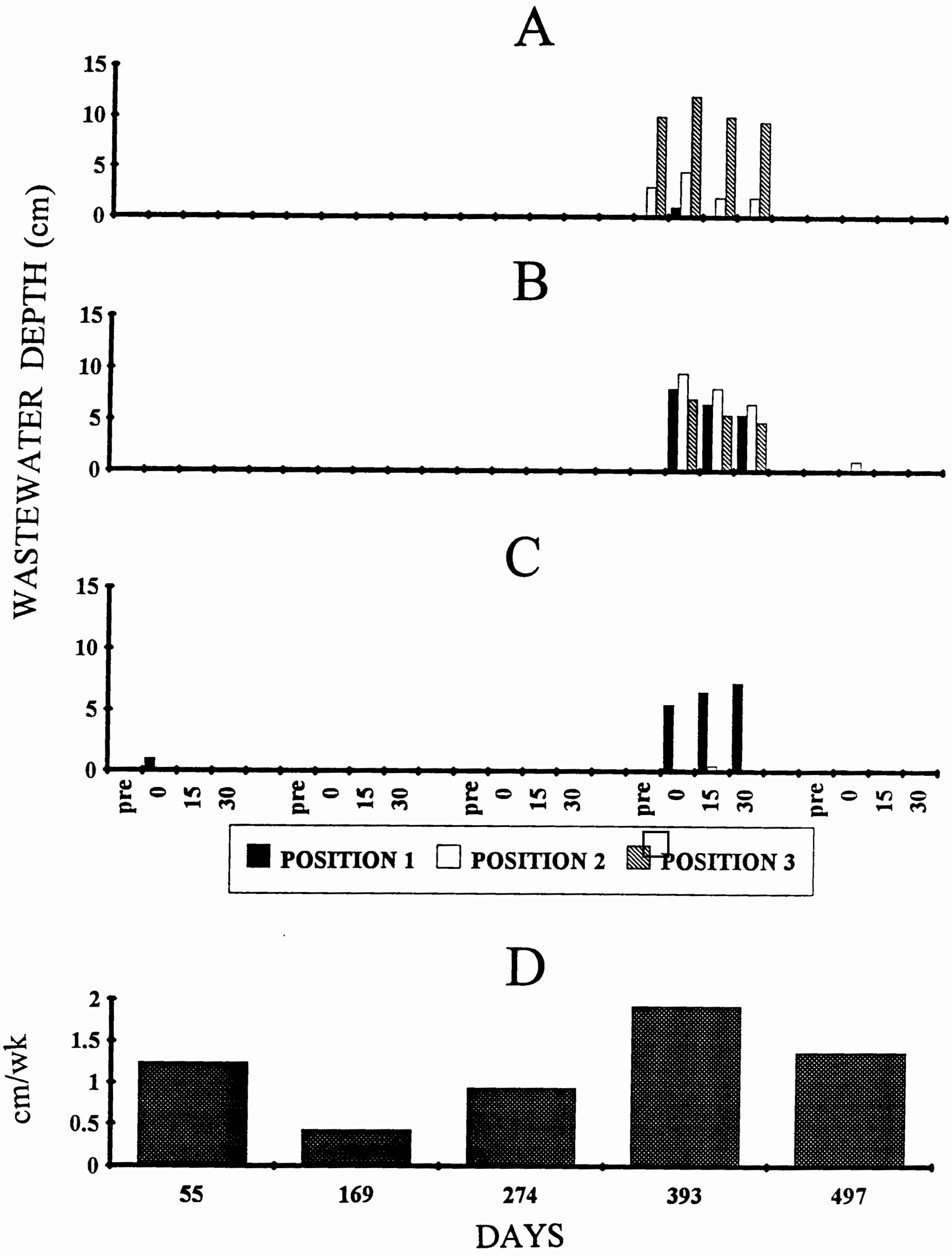

Figure 12. Wastewater level ponding at three locations within the trenches of LPP subfield 1 receiving treated effluent at (A) 0.075 gal ft^{-2} d^{-1}, (B) 0.1 gal ft^{-2} d^{-1}, and (C) 0.125 gal ft^{-2} d^{-1}. The weekly precipitation for the area is also given in D.

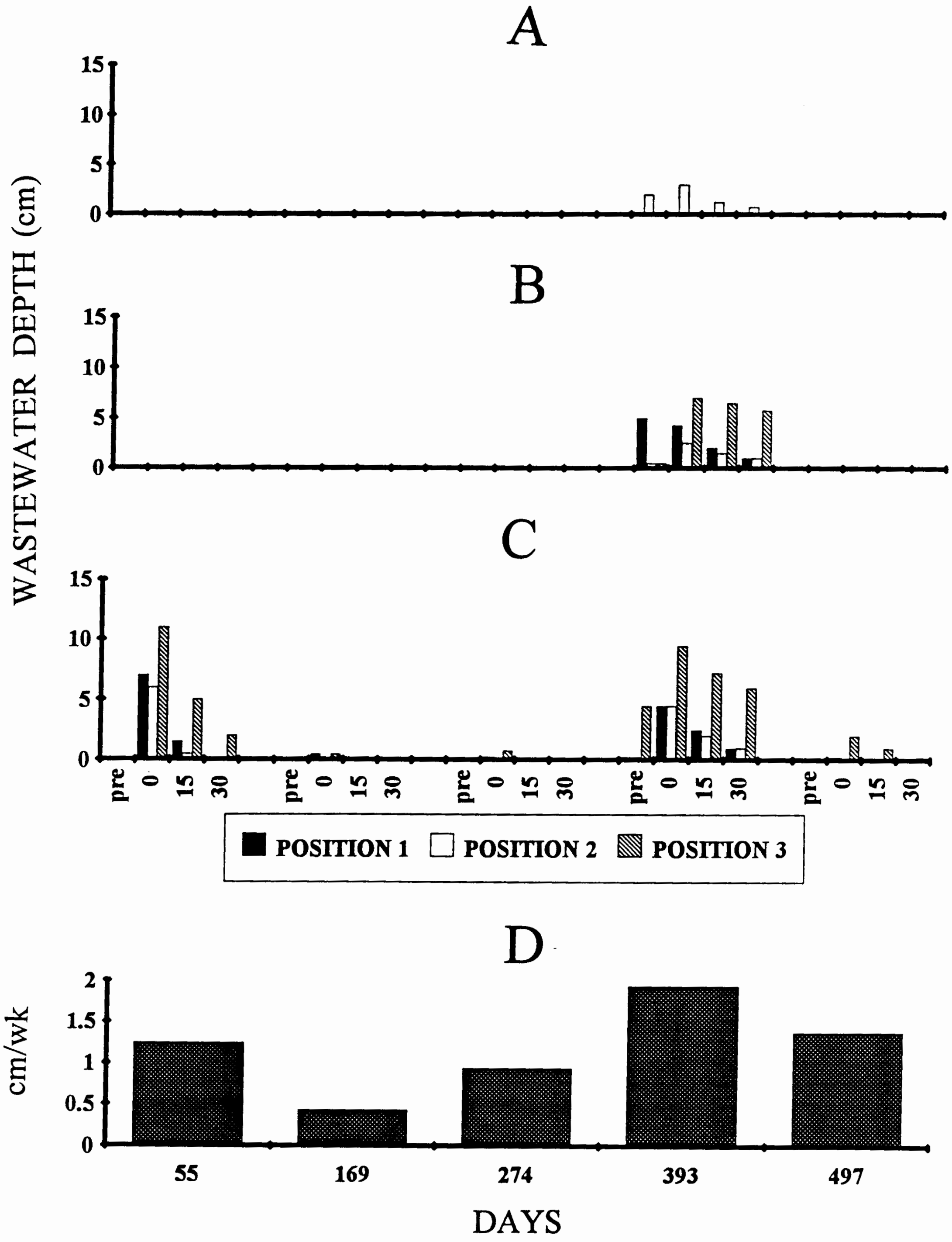

Figure. 13. Wastewater level ponding at three locations within the trenches of LPP subfield 3 receiving treated effluent at (A) 0.075 gal ft^{-2} d^{-1}, (B) 0.1 gal ft^{-2} d^{-1}, and (C) 0.125 gal ft^{-2} d^{-1}. The weekly precipitation for the area is also given in D.

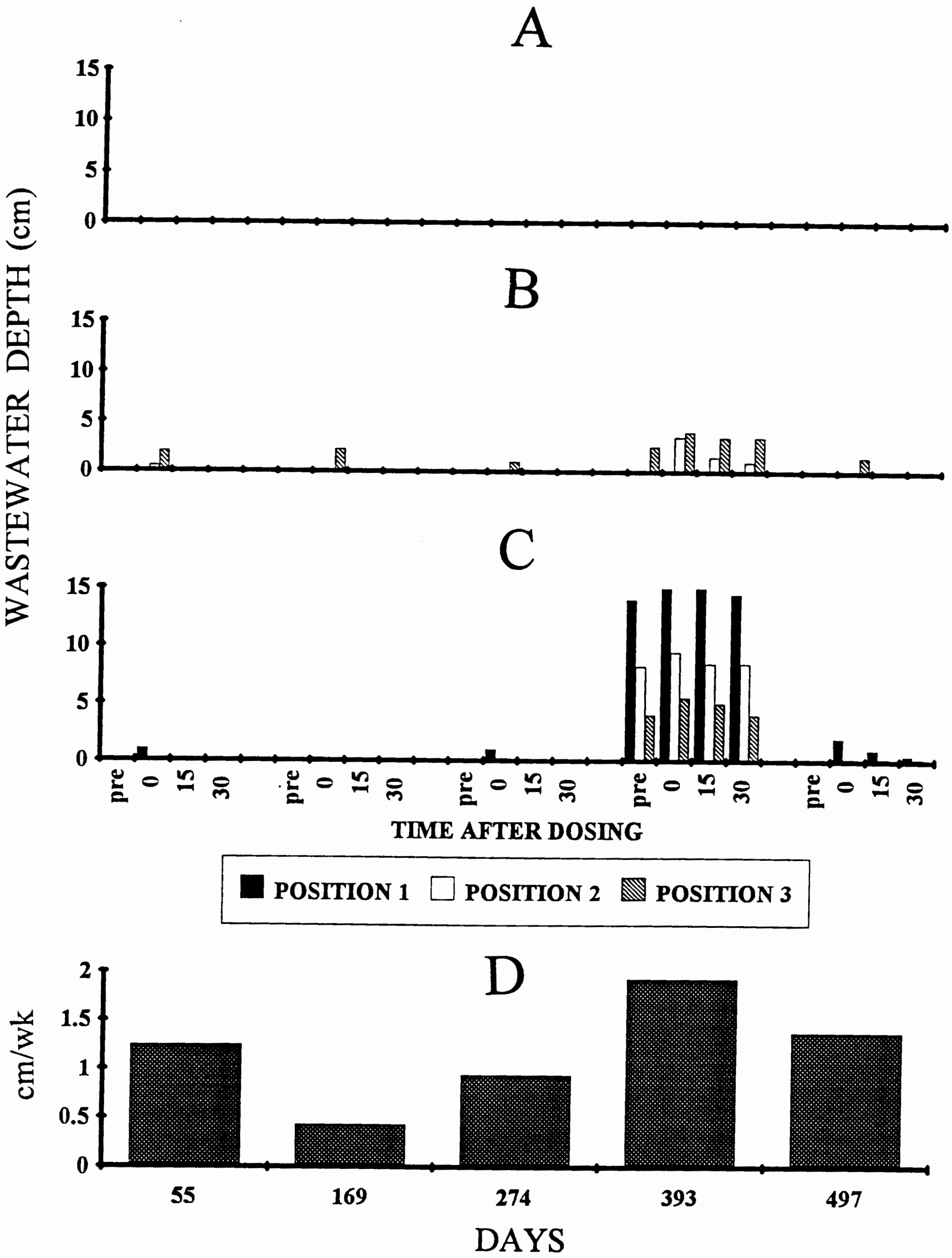

Figure 14. Wastewater level ponding at three locations within the trenches of LPP subfield 2 receiving untreated effluent at (A) 0.075 gal ft^{-2} d^{-1}, (B) 0.1 gal ft^{-2} d^{-1}, and (C) 0.125 gal ft^{-2} d^{-1}. The weekly precipitation for the area is also given in D.

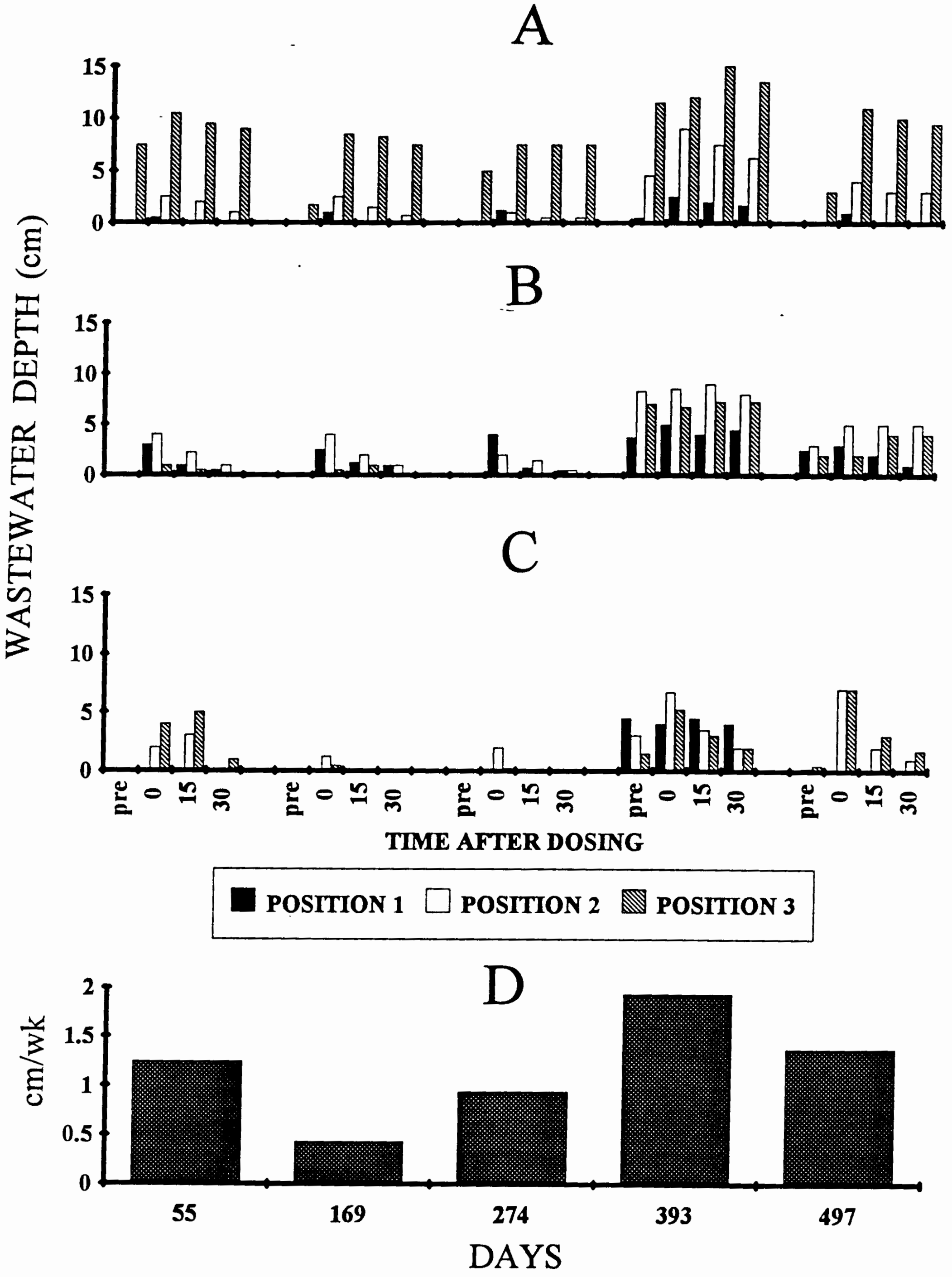

Figure 15. Wastewater level ponding at three locations within the trenches of LPP subfield 4 receiving untreated effluent at (A) 0.075 gal ft^{-2} d^{-1}, (B) 0.1 gal ft^{-2} d^{-1}, and (C) 0.125 gal ft^{-2} d^{-1}. The weekly precipitation for the area is also given in D.

During the early part of the study, the level of ponding in the trenches receiving the highest loading rate was not as extensive as that observed in other trenches. Toward the end of the study, there appeared to be an increase in the level of ponding in the trenches receiving 0.125 gal ft^{-2} d^{-1}. Ponding in the trenches for an extended period of time will increase the potential for the formation of a clogging mat which will reduce the infiltration rate from the trenches into the surrounding soil. Therefore, an increase in the ponding level could eventually result in a substantial reduction of the infiltration rate from the trenches, which could result in an eventual surfacing of effluent. Since more ponding was observed in the trenches of untreated effluent than treated effluent, the pretreatment of wastewater appears to maintain the capacity of the trenches for disposal of wastewater.

Soil Water Pressure Head: Zero or above zero soil water pressure head values indicate free water which is related to saturation. However, in soils with macropores (e.g., root channels, worm holes, large planar voids), zero or above zero soil water pressure head may not correspond with complete saturation. Instead, water may fill the macropores that intercept the cup of the tensiometer showing positive values while the matrix of the soil remains dry. This phenomenon has been observed by others evaluating septic systems using tensiometry and neutron scattering techniques (Martin, 1987; Weymann, 1989), and will be discussed later when soil water content data collected by time domain reflectometry technique are presented.

Except for the high loading rate (i.e., 0.125 gal ft^{-2} d^{-1}), soil water pressure head values at three distances below the bottom of the trenches of subfield 1 (Fig. 16) were relatively uniform and near zero except for a short time during the summer and fall (dry period). For subfield 2 receiving untreated wastewater, soil water pressure head values at all three distances below the bottom of the trenches were fairly uniform at near or above zero (Fig. 17). The uniformity of soil water pressure head values at intervals below the trenches indicate that the downward gradient is near 1 and that wastewater moves in a vertical direction.

The soil water pressure heads at three distances below the trench bottoms in subfields 3 and 4 receiving treated and untreated wastewater (Figs. 18 and 19), respectively, varied more with time than soil water pressure heads under subfields 1 and 2. Higher variation in soil water pressure head under these subfields could be due the thickness of the Bt and depth to the B/C or BC transitional horizons. For subfield 4 (Fig. 19), the soil water pressure head for 0.075 and 0.1 gal ft^{-2} d^{-1} loading rates remained below zero at 50 cm below the trench bottom and near zero at 10 and 30 cm below the trench bottom. These results correspond fairly well with the low K_{sat} values near the trench bottom which may restrict water movement. Saturated hydraulic conductivity at 50 cm below the trench bottom was considerably higher than the K_{sat} values of the BC horizon. This condition will result in unsaturated conditions as seen from the negative soil water pressure head values at 50 cm below the trench bottom. For the high loading rate of 0.125 gal ft^{-2} d^{-1}, near zero soil water pressure head was observed at all depths. For subfield 3 receiving treated effluent, soil water pressure head was more uniform and somewhat less than their corresponding values for subfield 4 which received untreated wastewater.

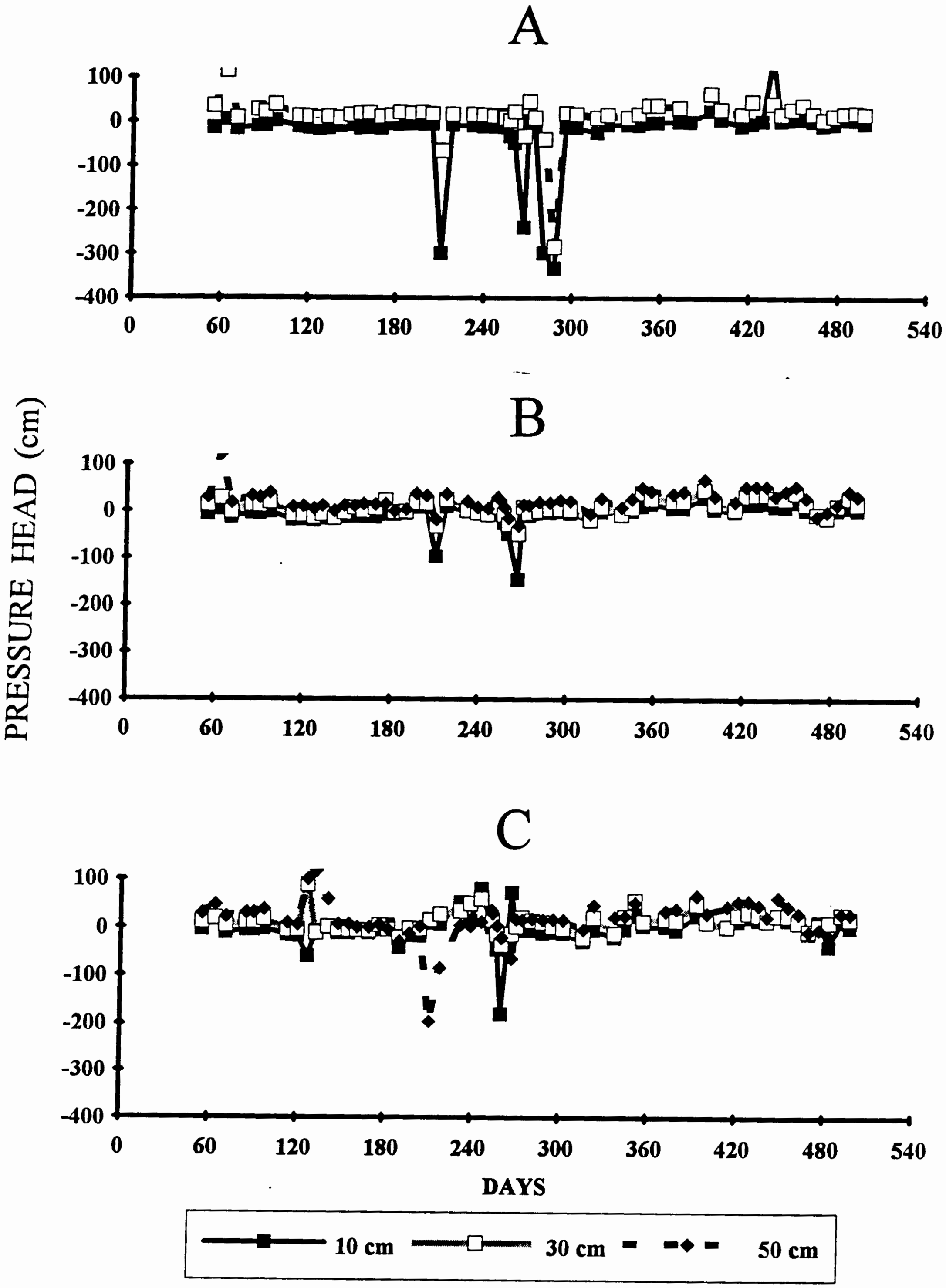

Figure 16. Soil water pressure head values for three depths below the trenches of the LPP subfield 1 receiving treated wastewater at (A) 0.075 gal ft^{-2} d^{-1}, (B) 0.1 gal ft^{-2} d^{-1}, and (C) 0.125 gal ft^{-2} d^{-1}.

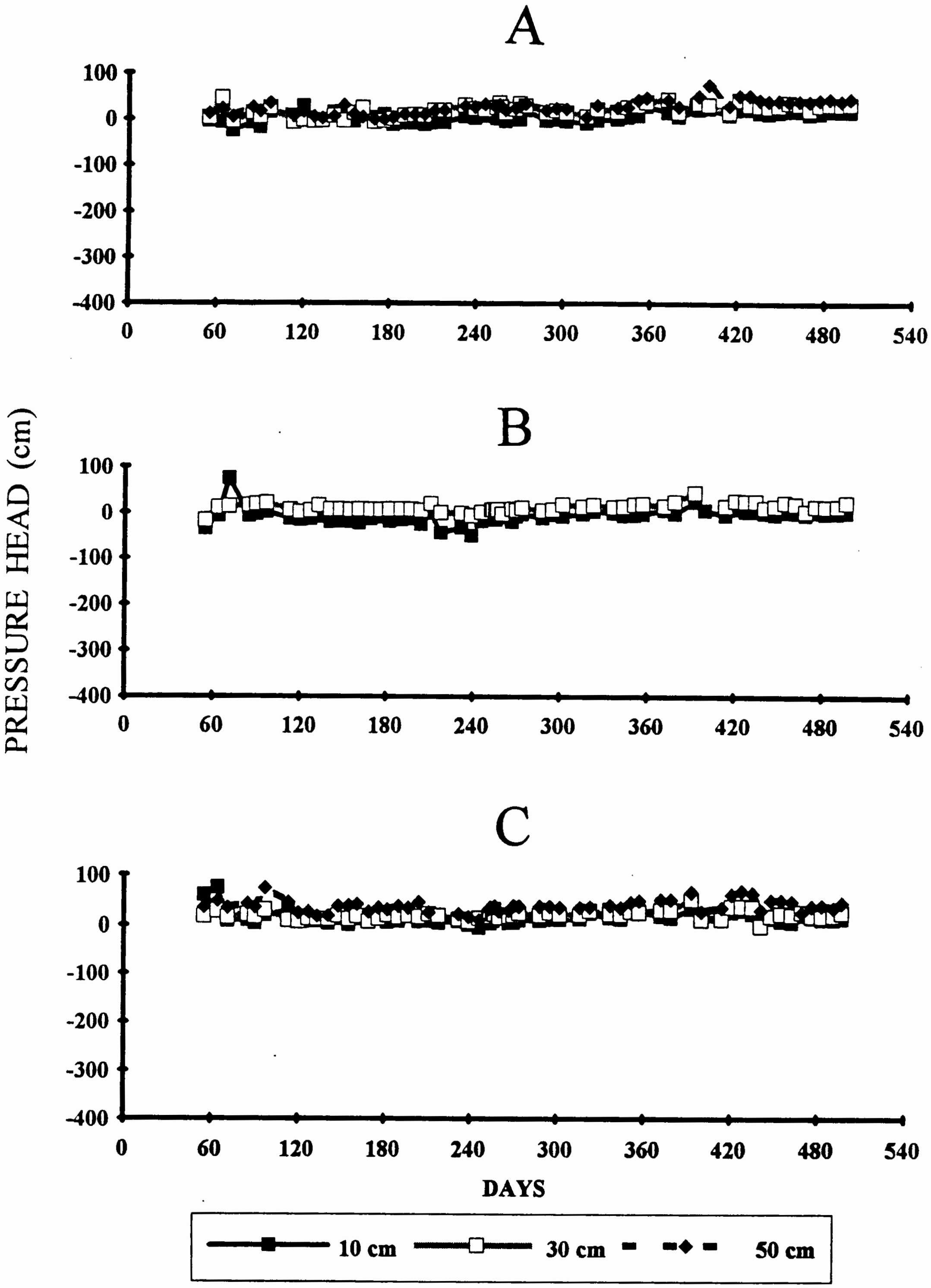

Figure 17. Soil water pressure head values for three depths below the trenches of the LPP subfield 2 receiving untreated wastewater at (A) 0.075 gal ft^{-2} d^{-1}, (B) 0.1 gal ft^{-2} d^{-1}, and (C) 0.125 gal ft^{-2} d^{-1}.

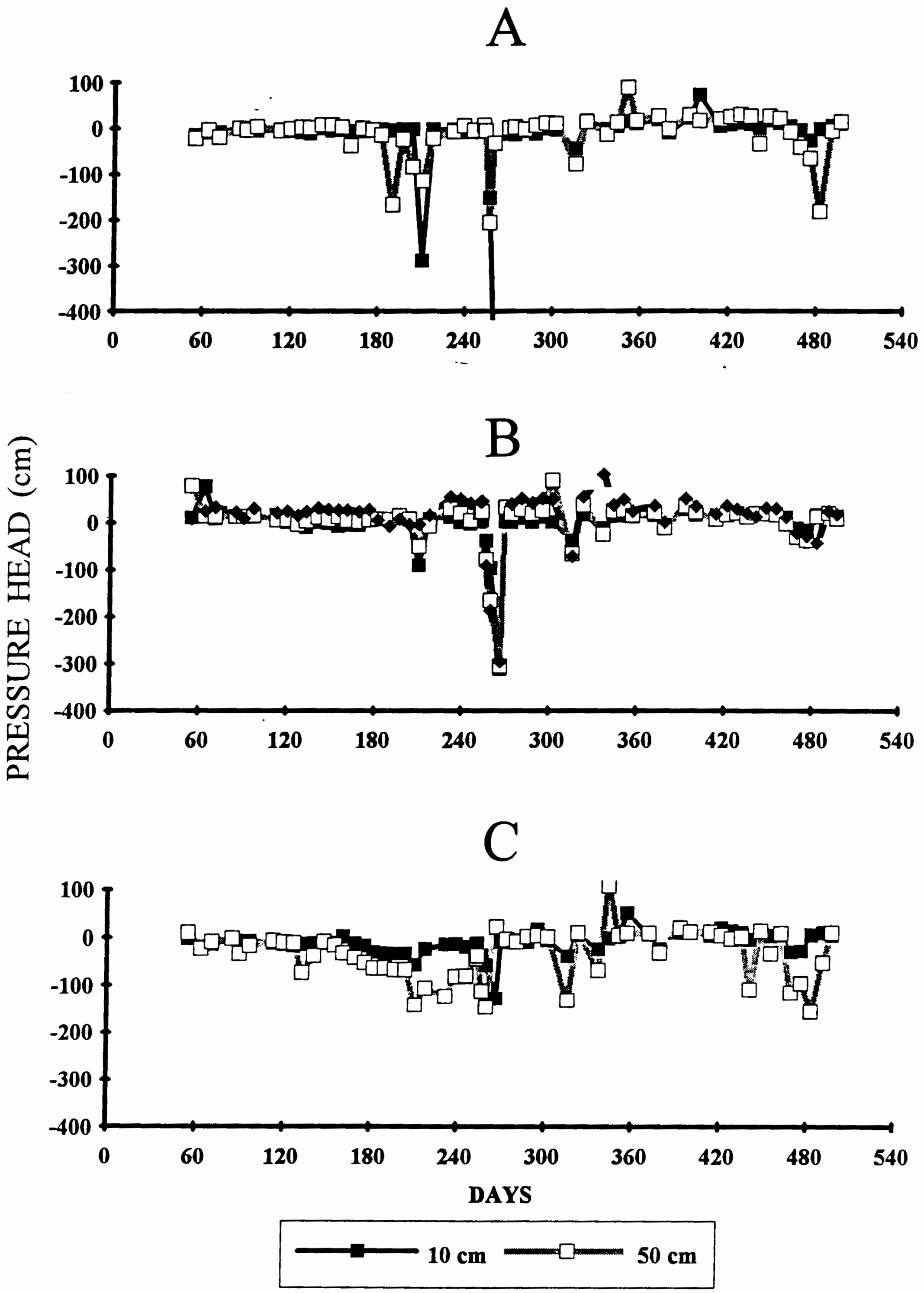

Figure 18. Soil water pressure head values for three depths below the trenches of the LPP subfield 3 receiving treated wastewater at (A) 0.075 gal ft^{-2} d^{-1}, (B) 0.1 gal ft^{-2} d^{-1}, and (C) 0.125 gal ft^{-2} d^{-1}.

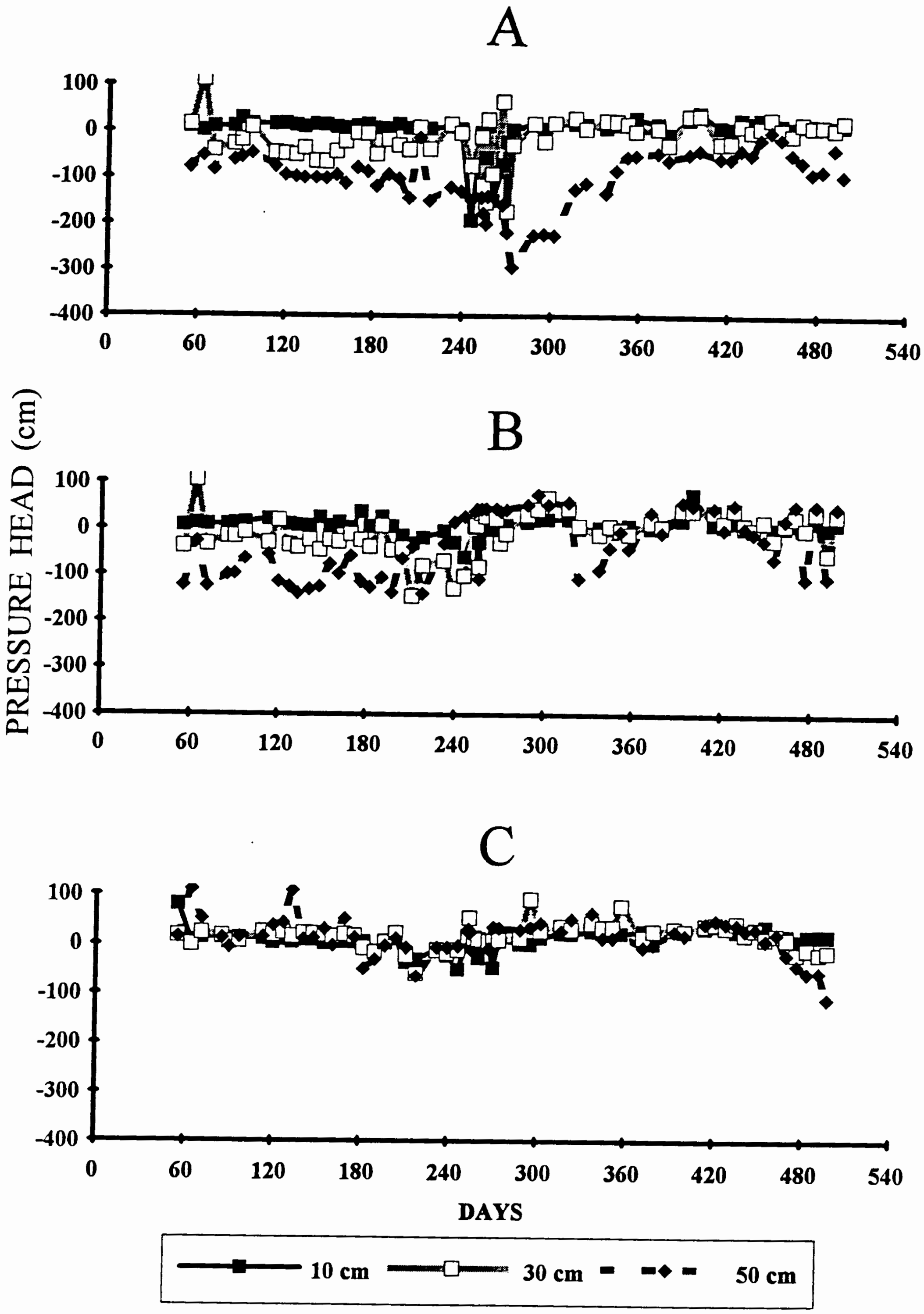

Figure 19. Soil water pressure head values for three depths below the trenches of the LPP subfield 4 receiving untreated wastewater at (A) 0.075 gal ft^{-2} d^{-1}, (B) 0.1 gal ft^{-2} d^{-1}, and (C) 0.125 gal ft^{-2} d^{-1}.

The soil water pressure head values for five depths at a distance of 15 cm from the side of the trenches for all four subfields showed considerable variation with depth and time. For subfield 1 (for data see West, 1994) with the deepest soil and the lowest K_{sat} at approximately 100 cm below the soil surface, unsaturated conditions were observed at 50 and 75 cm below the soil surface. At 100 to 150 cm below the soil surface, soil water pressure head was near or above zero. These results correspond well with the depth to the most restrictive layer at 125 to 150 cm depth interval. At subfield 2 (for data see West, 1994), which received untreated effluent, higher soil water pressure head values were recorded at 75 to 100 cm depth intervals. At 125 and 150 cm depths, soil water pressure head values were negative for most of the time of evaluation. Considering that the C horizon (saprolite) was at 125 cm depth, and that K_{sat} of saprolite was substantially higher than the K_{sat} of the BC horizon, downward movement of wastewater was impeded and more wetness was recorded at the bottom of the Bt and through the BC horizons.

For subfield 3, positive soil water pressure head values were recorded occasionally at 50 and 75 cm depth intervals (Fig. 20). For subfield 4, soil water pressure head values greater than zero were observed infrequently (Fig. 21). These combined with the results for the soil water pressure head directly under the trenches correspond well with the thickness and K_{sat} of various horizons and the depths to them. For example, the soil around the bottom of the trenches in subfield 4 had a relatively low K_{sat} resulting in ponding of wastewater in the trenches and little lateral movement of wastewater from the trenches.

Overall, the results for soil water pressure heads at various depths directly under each trench and at a distance of 15 cm from the trench wall agree with the ponding data for the respective trench in each subfield, and with the K_{sat} at various depths. It also appears that treated wastewater moves more freely through the profile and localized saturations at certain depth intervals may be associated with the quality of percolating soil water.

Soil Water Content: Soil water content was determined at a distance of 15 cm from the trench walls by the TDR technique from July 1993 through May 1994. As mentioned previously, four pairs of TDR rods from 15 to 60 cm length were used and the volumetric soil water content values for 0 to 15, 15 to 30, 30 to 45, and 45 to 60 cm depth intervals were calculated using the measured soil water content from the soil surface to the bottom of the corresponding pairs of rods.

The soil water content for each depth interval in subfield 1 with 30-cm thick A and BA horizons, and receiving treated wastewater, remained below 0.4 $m^3\ m^{-3}$ for all three loading rates (Fig. 22). For all loading rates, the 45 to 60 cm depth interval was the wettest for most times. This corresponds with wastewater moving from the bottom of the trenches into the surrounding soil materials as shown by little ponding in the trenches. For subfield 2, which received untreated effluent, soil water content at 30 to 45 and 45 to 60 cm depth intervals approached 0.5 $m^3\ m^{-3}$ (Fig. 23). Based on the soil water characteristic and bulk density data, saturation in the soils found in subfields 1 and 2 was a little more than 50%. The lowest K_{sat} value for both subfields was approximately 0.12 cm d^{-1} at 100 cm depth. The soil in subfield 2 area was slightly shallower than the soil in subfield 1. The difference between the soil water content values for the two subfields, however, cannot be explained by the differences in the soil thickness. Instead, it

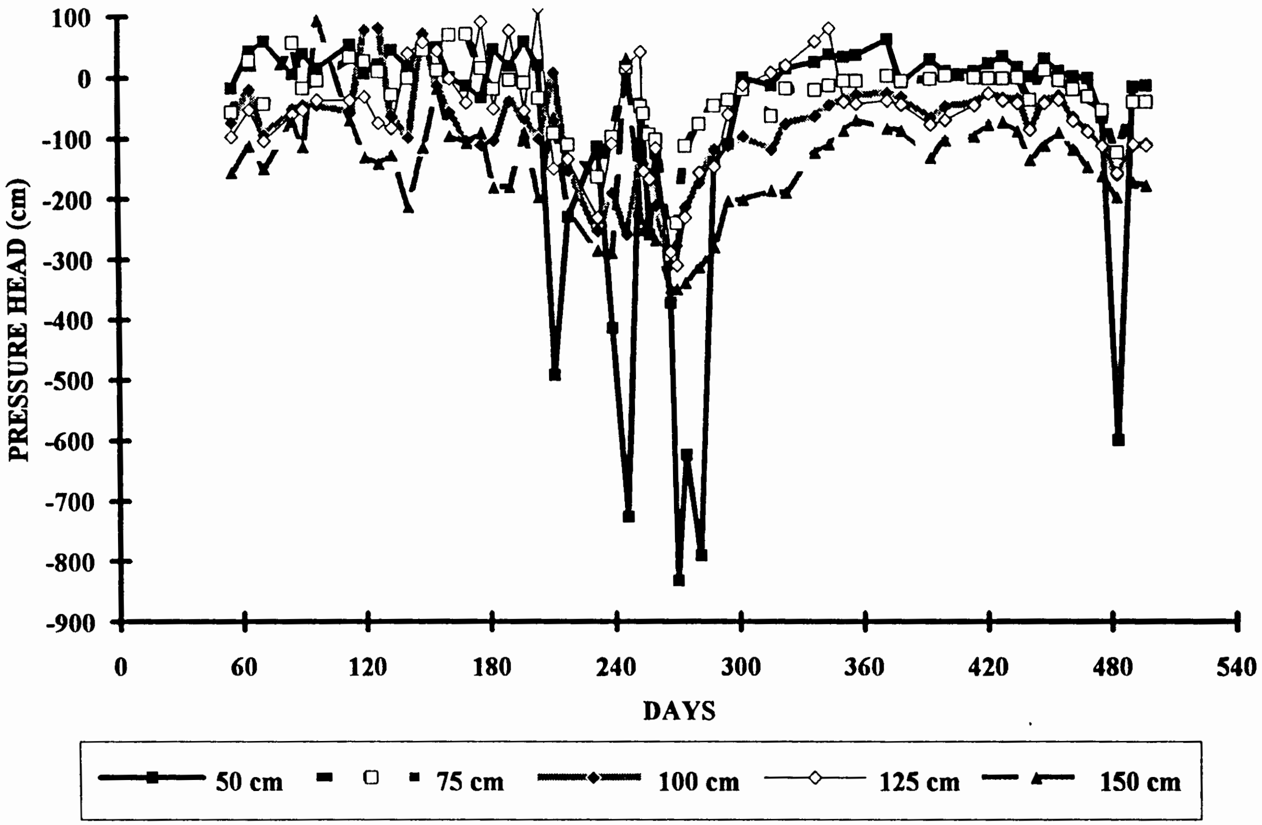

Figure 20. Soil water pressure head values for five depths adjacent to the middle trench in the LPP subfield 3 receiving treated wastewater.

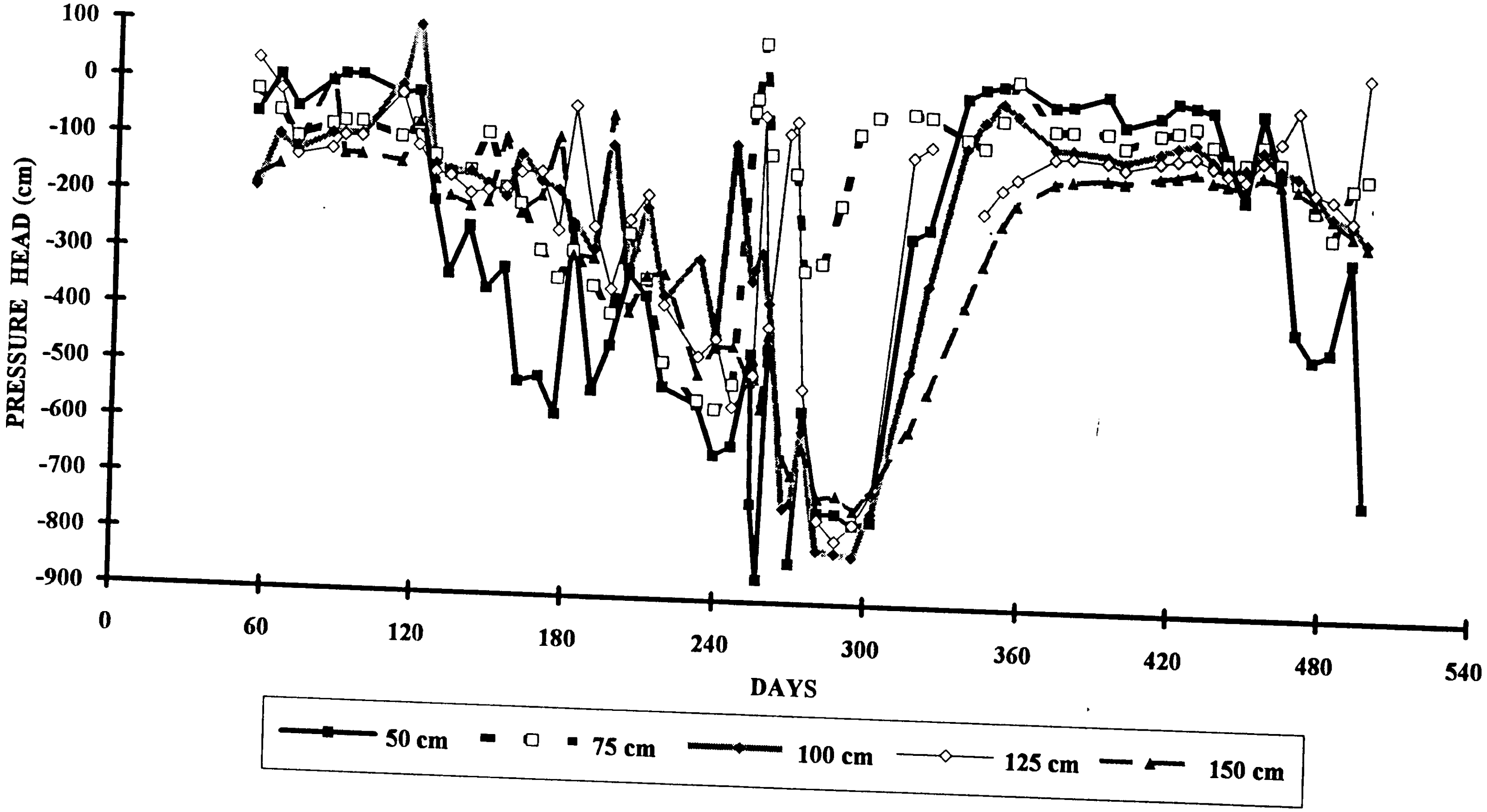

Figure 21. Soil water pressure head values for five depths adjacent to the middle trench in the LPP subfield 4 receiving untreated wastewater.

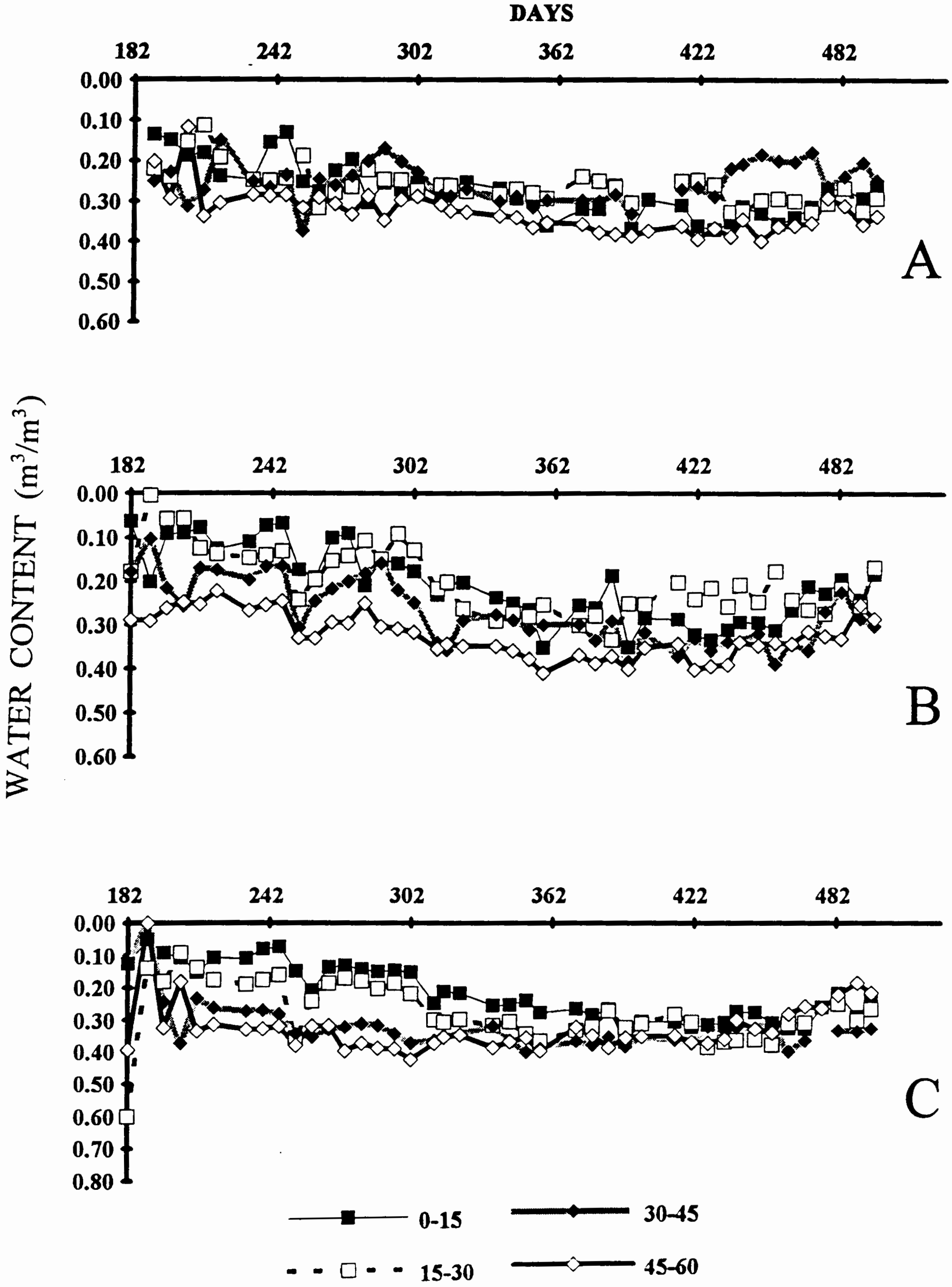

Figure 22. Soil water content for four depth intervals in the LPP subfield 1 receiving treated wastewater at (A) 0.075 gal ft^{-2} d^{-1}, (B) 0.1 gal ft^{-2} d^{-1}, and (C) 0.125 gal ft^{-2} d^{-1}.

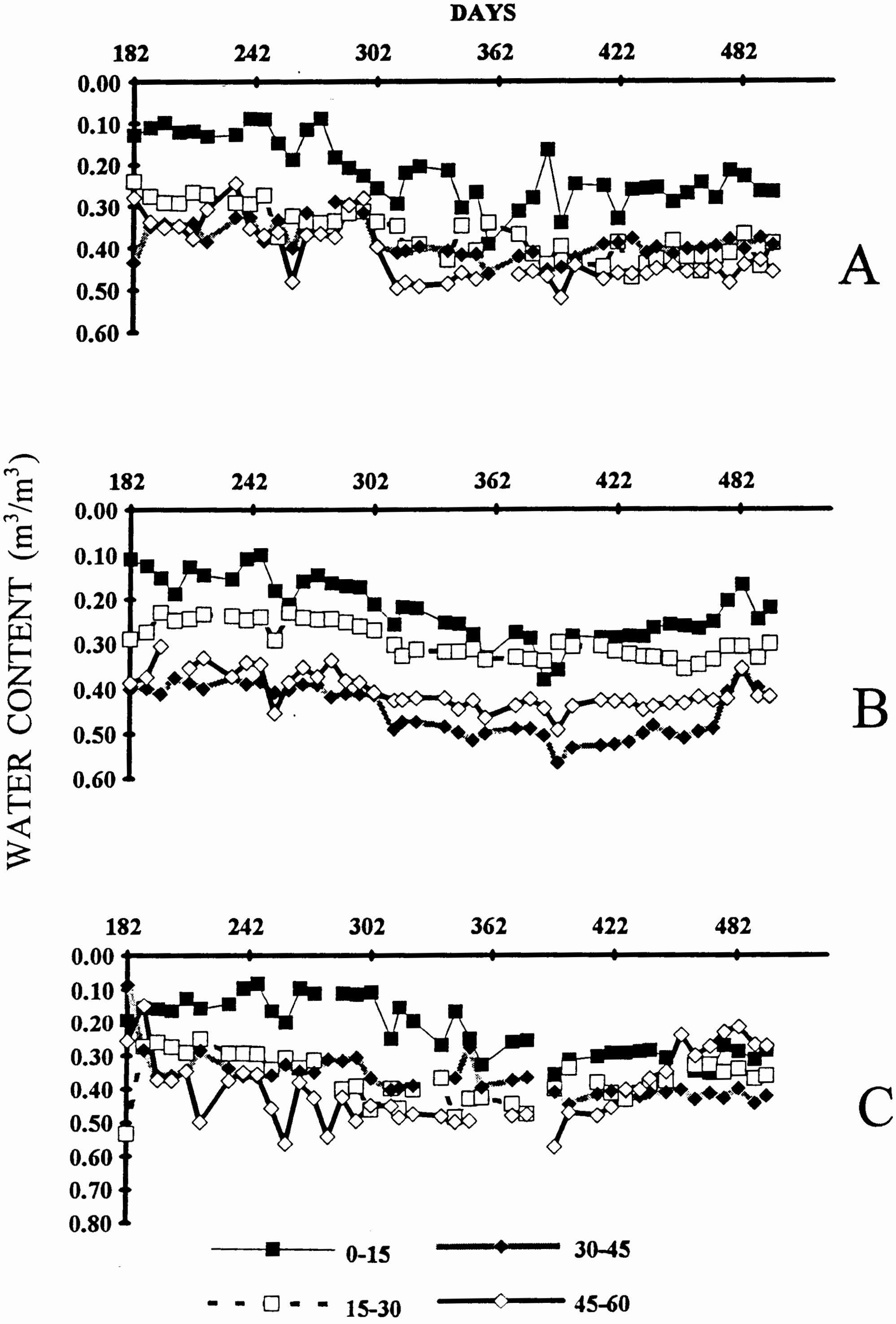

Figure 23. Soil water content for four depth intervals in the LPP subfield 2 receiving untreated wastewater at (A) 0.075 gal ft^{-2} d^{-1}, (B) 0.1 gal ft^{-2} d^{-1}, and (C) 0.125 gal ft^{-2} d^{-1}.

appears that untreated effluent entering the soil from the trenches resulted in a gradual slow down of water movement, perhaps due to partial clogging of the pores, and an eventual increase in the soil water content near the bottom of the trenches.

For subfield 3, receiving treated wastewater, soil water content at all four depth intervals was less than 0.5 $m^3 m^{-3}$ for the entire measurement period (Fig. 24). For the untreated wastewater, the results for subfield 4 (Fig. 25) indicate a higher soil water content at deeper depths than the corresponding depth for the treated effluent. In fact, calculated soil water content at 45 to 60 cm depth interval exceeded 0.6 $m^3 m^{-3}$ for the first part of the measurement period. Although water content greater than the actual total porosity of the soil may be due to the inherent error associated with weighted averaging of the soil water content for various depth intervals, the 45 to 60 cm depth interval was determined to be wetter than the other depth intervals. Higher soil water content observed at 30 to 45 and 45 to 60 depth intervals correspond with the ponding in the trenches and the nature of the soils and the untreated wastewater used in this study.

Overall, the results indicate that wastewater treated by an ATU infiltrated into the soil and moved away from the trenches much faster than untreated effluent. Comparing the soil water content data with the overall soil water pressure head measurements shows discrepancies. However, as mentioned earlier, direct comparison of the volumetric soil water content and measured soil water pressure head is made difficult because of the potential impact of macropores on soil water pressure head measurement.

Drip Systems

Ponding in the Trenches: Wastewater ponding was monitored at one location within three of the subfields and at three locations in one of the trenches of the subfield receiving untreated effluent at 0.2 gal $ft^{-2} d^{-1}$ (see Fig. 4). For the treated wastewater, no ponding was observed in the observation well near the drip line for the 0.2 gal $ft^{-2} d^{-1}$, while for the 0.1 gal $ft^{-2} d^{-1}$ some ponding had started to occur toward the end of the study period (Fig. 26). For the untreated wastewater, however, ponding was relatively high for the 0.1 gal $ft^{-2} d^{-1}$ and somewhat variable for the 0.2 gal $ft^{-2} d^{-1}$ application rate. The quantity of ponding started to increase toward the end of the study period for the 0.2 gal $ft^{-2} d^{-1}$. Because of our observation of higher ponding for the 0.1 gal $ft^{-2} d^{-1}$ than the 0.2 gal $ft^{-2} d^{-1}$ for both treated and untreated wastewater early in the study, the control panels and the four subfields were checked to assure that each system was receiving the design loading rate. Since wastewater ponding was monitored at only one location within three of the subfields and at three neighboring locations within the other subfield, we believe higher ponding in the observation wells installed near the drip lines receiving 0.1 gal $ft^{-2} d^{-1}$ was due to the nonuniformity of wastewater distribution and excessive drainage at the end of a dosing cycle from orifices that were at a lower elevation along the laterals. Note that the trenches were not installed on contour, and that the drip lines were placed inside each trench and covered with soil without any leveling. This method of installation may result in substantial elevation differences among the orifices. The uniformity of flow from orifices and the potential free drainage from some orifices along the laterals will be discussed later.

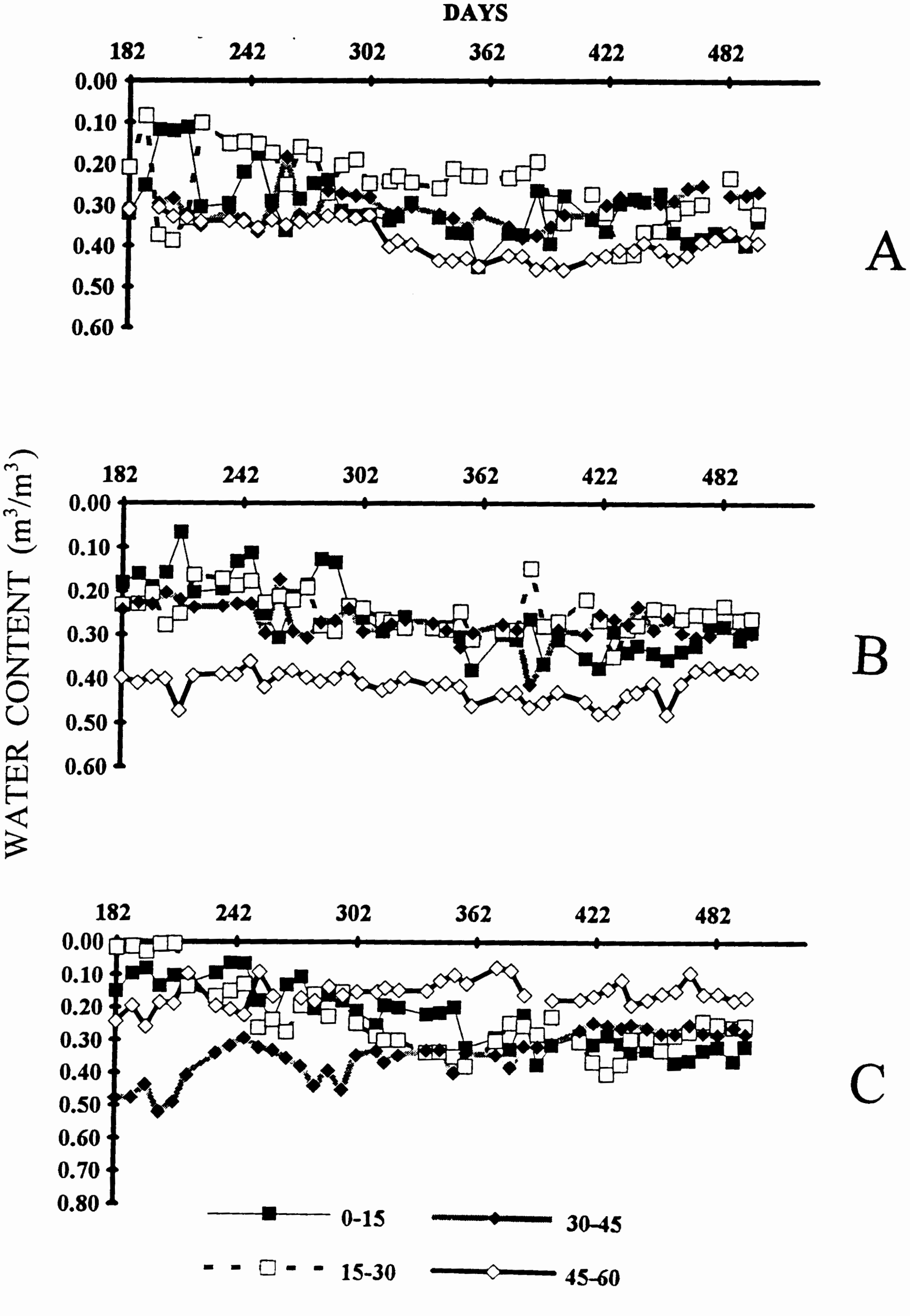

Figure 24. Soil water content for four depth intervals in the LPP subfield 3 receiving treated wastewater at (A) 0.075 gal ft^{-2} d^{-1}, (B) 0.1 gal ft^{-2} d^{-1}, and (C) 0.125 gal ft^{-2} d^{-1}.

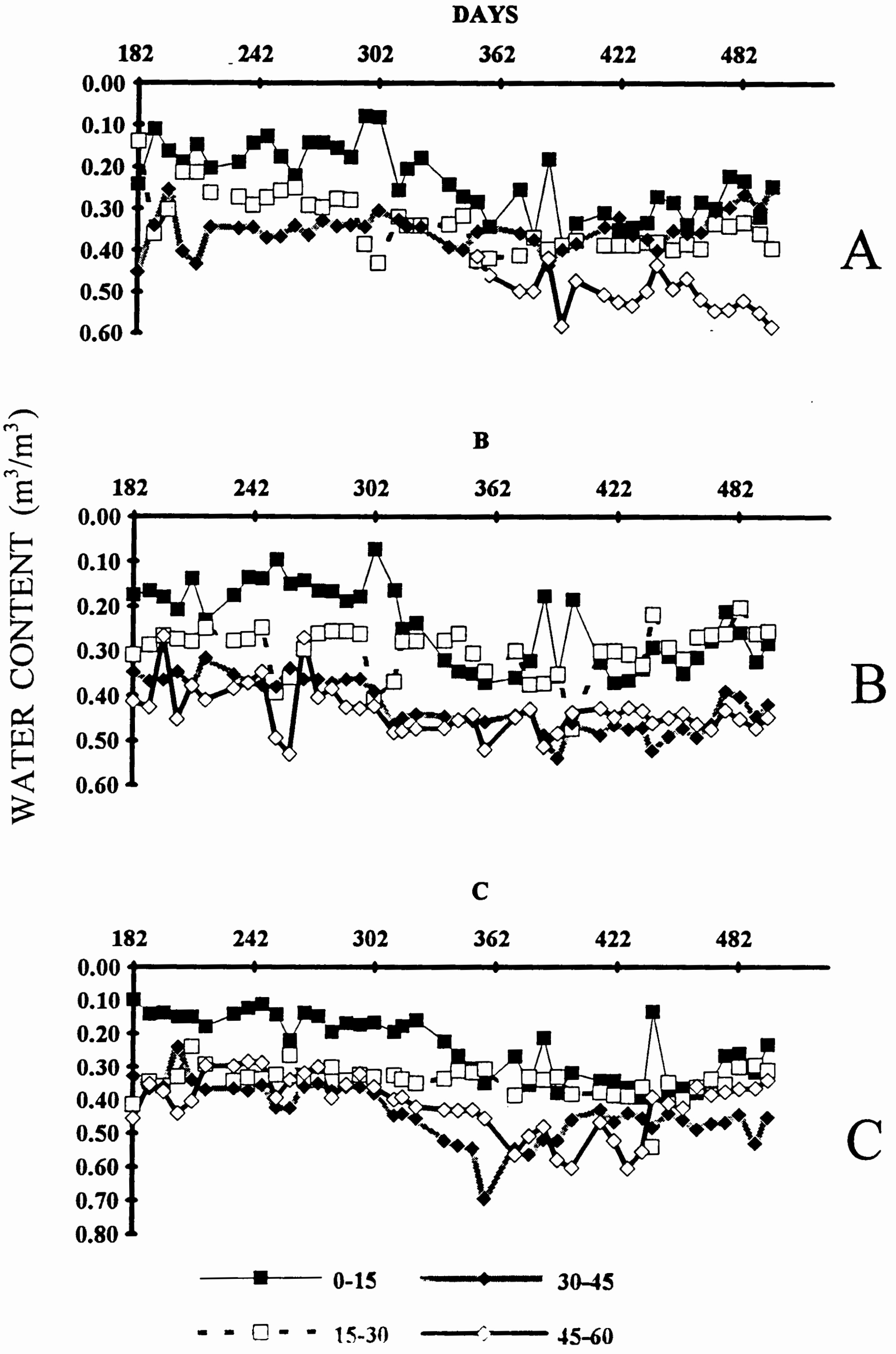

Figure 25. Soil water content for four depth intervals in the LPP subfield 4 receiving untreated wastewater at (A) 0.075 gal ft^{-2} d^{-1}, (B) 0.1 gal ft^{-2} d^{-1}, and (C) 0.125 gal ft^{-2} d^{-1}.

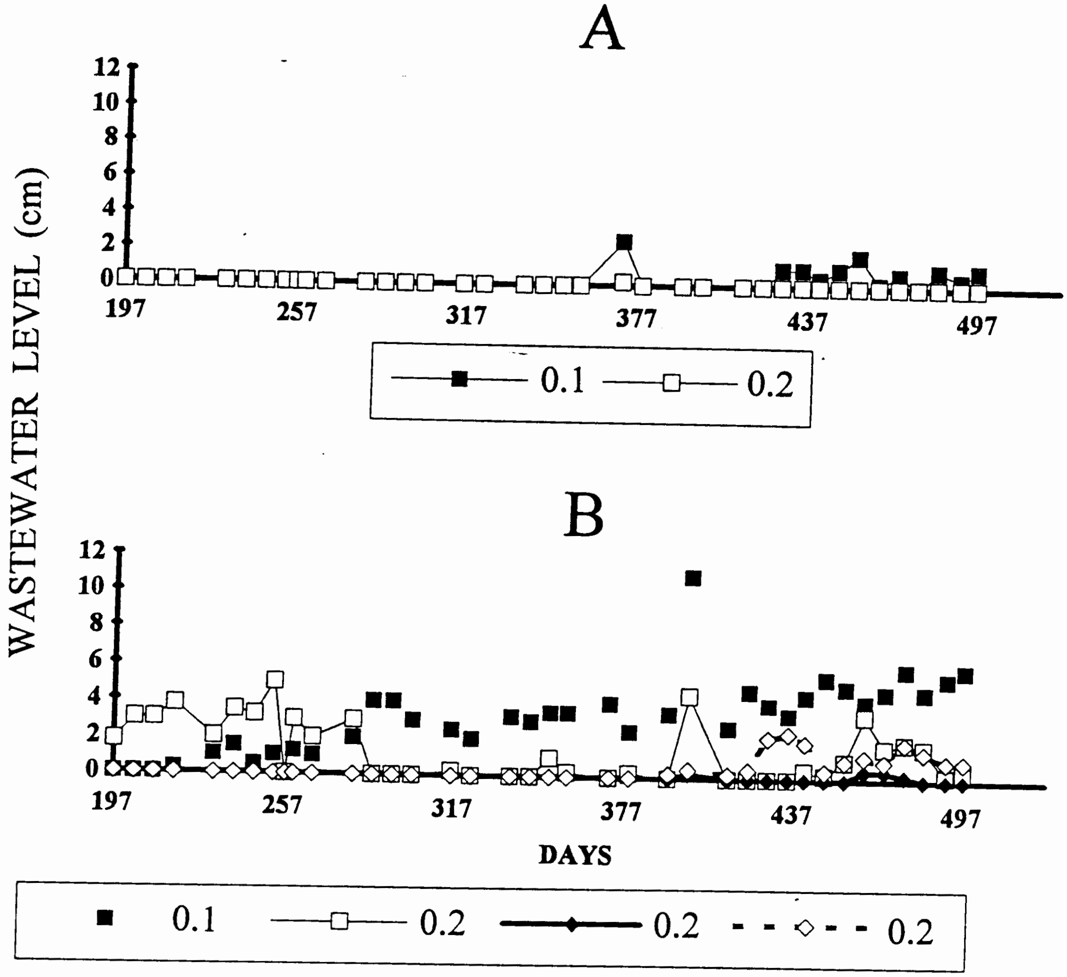

Figure 26. Wastewater ponding in the observation wells near the drip lines receiving (A) treated wastewater and (B) untreated wastewater at 0.1 and 0.2 gal ft^{-2} d^{-1}. For the untreated wastewater subfield, measurements were made at three locations next to a lateral line receiving wastewater at the rate of 0.2 gal ft^{-2} d^{-1}.

Soil Water Pressure Head: Soil water pressure head values at three depths outside the drainfield areas of the subfields receiving treated and untreated effluent were less than zero for most of the study period. Figure 27 presents the background data for two locations near the 0.1 gal ft^{-2} d^{-1} drip fields at this site (see Fig. 4). As we discussed earlier, theoretically, zero or positive soil water pressure head indicates saturated conditions. However, tensiometers installed in highly structured soils, or in soils with macropores such as root channels, a zero or positive soil water pressure head may not correspond with total saturation. Instead, water moving in macropores may result in high soil water pressure head readings made by tensiometers.

Soil water pressure head around the emitters of the drip systems receiving treated and untreated effluent at 0.1 gal ft^{-2} d^{-1} was near zero for most of the study period (for data see West, 1994). Adjacent to the trenches, soil water pressure head at different depths was also near zero for this loading rate. For 0.2 gal ft^{-2} d^{-1}, soil water pressure head around the emitters for both treated and untreated wastewater was less than zero for a relatively short period. Figure 28 presents the soil water pressure head data for four positions around one emitter for this loading rate. The four positions were at the center (labeled 0C-0-0), 10 cm below the emitter (labeled 0C-10-0), 20 cm to the right and 10 cm below the emitter (labeled 20R-10-0), and 20 cm to the left and 10 cm below the emitter (labeled 20L-10-0). Outside of the trenches, soil water pressure head showed dryer conditions for this loading rate (Fig. 29). Although soil water pressure head results may be surprising, they are consistent with higher ponding in the trenches for the 0.1 gal ft^{-2} d^{-1}. As indicated earlier, the differences in the soil water pressure head values for the two application rates are perhaps due to a lack of uniform distribution resulting from uneven elevation of the orifices along the laterals.

Comparing the results presented in Figs. 28 and 29 for soil water pressure heads inside and outside the trenches, respectively, it appears that the treated wastewater moved away from the trenches at a faster rate than the untreated wastewater. Comparing the background soil water pressure head values with the measured values for corresponding depths inside the drainfield area, it appears that the wastewater applied to the soil had a slight influence on soil water pressure head inside the drainfield area.

Wastewater Quality

Wastewater samples were collected from the LPP and drip systems on three dates from January to May, 1994. Each sample was analyzed for BOD_5, COD, and fecal coliform and the results are given in Table 3. Pretreatment by the ATU had a pronounced effect on the BOD_5, COD and fecal coliform. The BOD_5 and COD of the samples collected from the treated and untreated drip fields were lower than their corresponding values for the treated and untreated LPP systems. The corresponding results for fecal coliform for the drip and LPP systems receiving treated and untreated wastewater, on the other hand, were not significantly different.

The differences in the BOD_5 and COD for the LPP and drip systems were due to the filtering of the treated wastewater, which had removed some of the organic particles. It appears that the filtering system had little influence on the fecal coliform in wastewater. The BOD_5, COD, and fecal coliform for the untreated wastewater were within the range of values reported for untreated

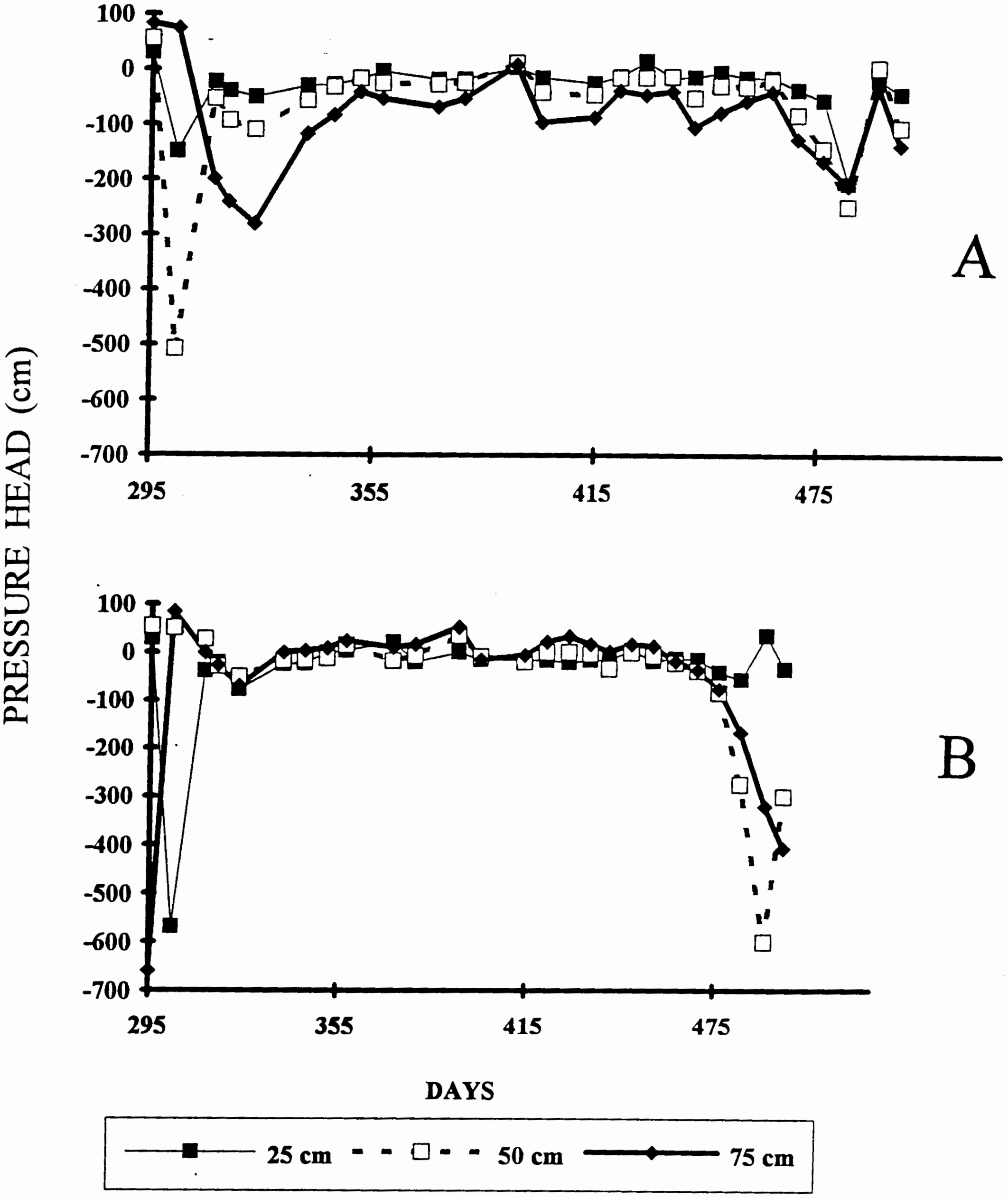

Figure 27. Background soil water pressure head values for three depths outside the 0.1 gal ft^{-2} d^{-1} subfields for (A) treated wastewater and (B) untreated wastewater.

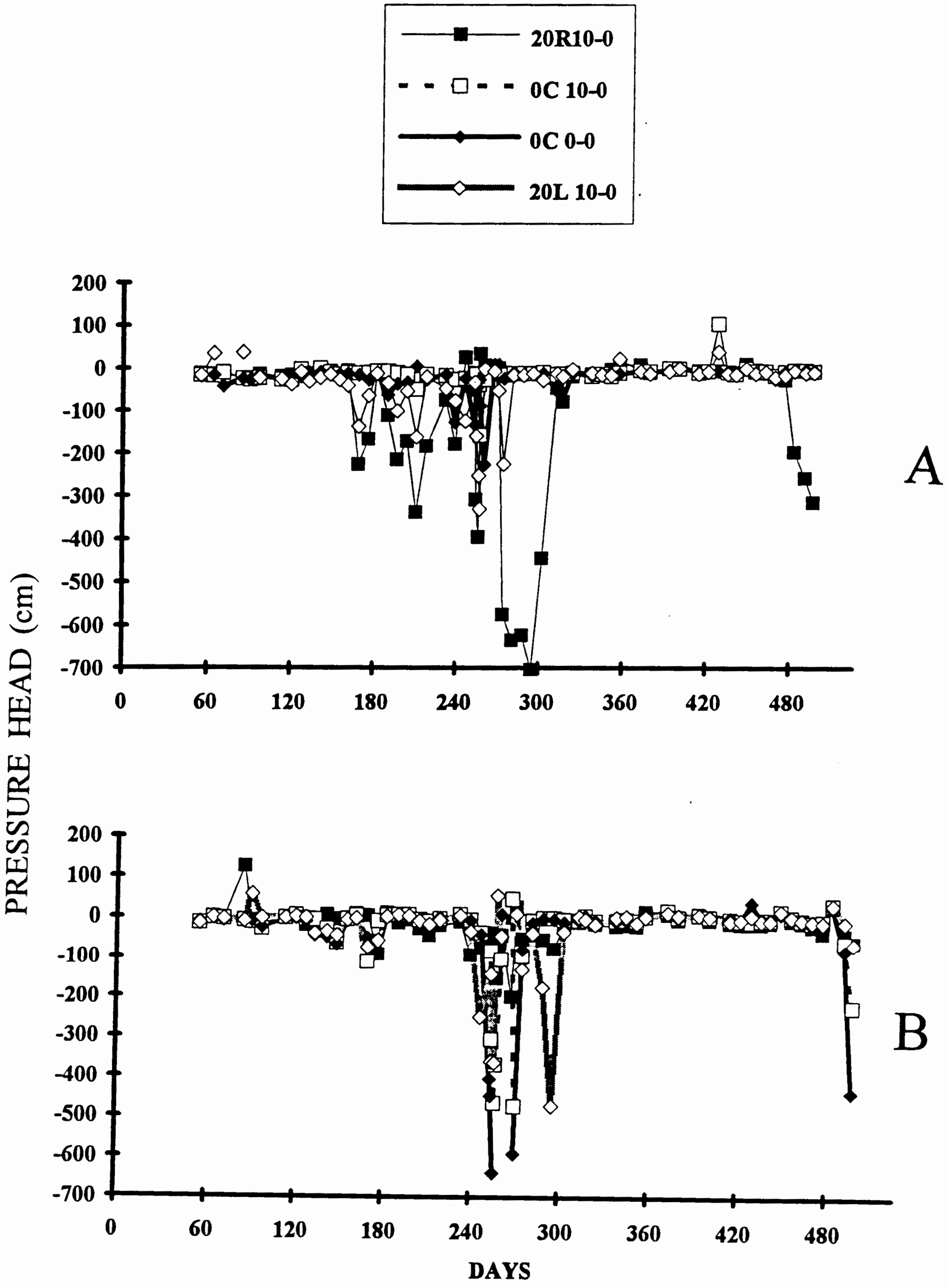

Figure 28. Soil water pressure head values for four locations around one emitter in the 0.2 gal ft^{-2} d^{-1} subfields for (A) treated wastewater and (B) untreated wastewater. For locations of the tensiometers around the emitter see Fig. 5.

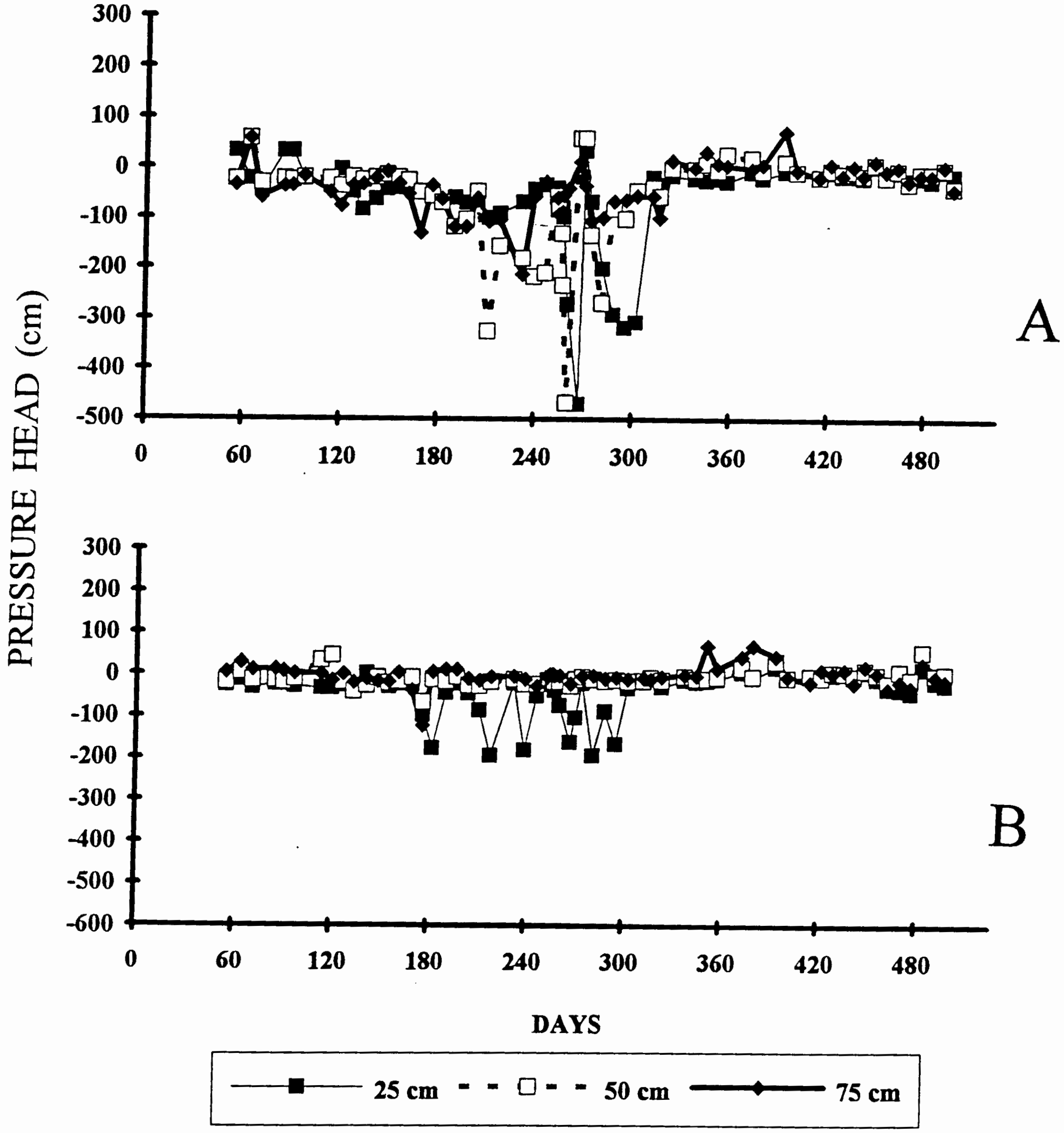

Figure 29. Soil water pressure head values for three depths near the drip line in the 0.2 gal ft^{-2} d^{-1} subfields for (A) treated wastewater and (B) untreated wastewater.

Table 3. Selected properties of wastewater samples collected from the LPP and drip systems at the Wake County site.

COLLECTION AREA	BOD$_5$	COD	FECAL COLIFORM
	mg L^{-1}		count per 100 mL
LPP			
Treated	14.8 - 42.2	43.2 - 83.1	920 - 4,000
Untreated	159 - 326	292 - 428	210,000 - 6,600,000
DRIP			
Treated	8.34 - 27.4	34.1- 51.7	920 - 4,900
Untreated	90.6 - 133	265- 409	323,000 - 573,000

municipal wastewater (EPA, 1992). As for treated wastewater, the values for all three parameters were also within the ranges given for various treatment processes for conventional wastewater treatment (EPA, 1992), and for an ATU reported by Carlile (1994).

The results for the treated and untreated wastewater also corresponded with our observation of the functioning of the flow meters on each lateral line of the LPP systems and buildup of materials inside the drip lines by the end of the study. [NOTE: To our knowledge, there is no flow meter that is commercially available for measuring the flow rate of untreated effluent. We used regular flow meters that are used for water which had the potential not to function properly for wastewater.] At the end of the study period, all of the drip lines installed at both sites were removed from the ground. Two or three small sections of each of the drip lines installed at the Wake County site were cut open and inspected. Although solid accumulation was not significant for both the treated and untreated wastewater, there were more solids that were accumulated on the sidewall of the lines for untreated wastewater than the ones for the treated wastewater. For the flow meters, the total amount of flow into the lines receiving treated effluent corresponded well with the design loading rate for individual lines (for data see West, 1994). For the untreated wastewater, on the other hand, the measured values obtained from the flow meters were erratic, and had no correspondence with the design loading rates. Inspection of the flow meters at the end of the study revealed that the screen in the flow meters were partially blocked by the suspended solids present in the untreated effluent. This confirms that the suspended solids in the untreated effluent can result in the blockage of the holes on the laterals of LPP and orifices of the drip lines. The suspended solids can also accumulate at the trench bottom or block the pores within the soil under the system. Both these can result in accumulation of wastewater in the trenches and saturated conditions in the soil under and around the trenches.

Chatham County Site

Soil and Site Characterization

Soil Profile Description: The soil profile at various locations within the drip and spray fields were evaluated using the hand-auger boring technique. In the spray field area, the soil had a 15-cm thick A horizon underlain by a 70-cm thick very sticky, very plastic Bt1 and Bt2 horizons. The C horizon had a massive structure and was firm, sticky and plastic. This soil was placed in the Creedmoor soil series (a clayey, mixed, thermic Aquic Hapludult). In the drip field area, two different soils, a Creedmoor in the upper part of the field and a White Store (a fine, mixed, thermic Hapludalf) in the lower part of the field were identified. In the upper part, the Bt1 horizon started at about 12 cm below the surface and the Bt2 horizon extended to 90 cm depth. In the lower part of the drip field, the combined Ap and E horizons were 30 cm thick, and the C horizon started at about 90 cm below the soil surface. The detailed profile descriptions for all three areas are presented by West (1994).

Particle Size Distribution: Particle size distribution for the drip and spray fields are given in Table 4. The sand content of the upper 20 cm in the drip area was greater than 66% while in the spray field only the top 10 cm had a sandy texture. In the drip field, the clay content at depths greater than 20 cm was 40 to 50%. For the spray field, the clay content in the upper 60 cm was less than 44.5%.

Table 4. Particle size distribution with depth for the spray and drip fields at the Chatham County site.

		DEPTH INTERVALS, cm							
FIELD	PARTICLE SIZE	0-10	10-20	20-30	30-40	40-50	50-60	60-70	70-80
		%							
Spray	Sand	70.5	44.5	44.5	43.7	43.7	28.4	ND	ND
	Silt	18.0	17.5	20.4	21.1	20.7	27.1	ND	ND
	Clay	11.5	38.0	35.1	35.2	35.6	44.5	ND	ND
Drip	Sand	70.6	66.4	30.8	34.3	27.3	23.8	31.3	13.9
	Silt	ND	18.6	19.1	22.4	22.1	30.1	28.5	45.0
	Clay	ND	15.0	50.1	43.3	50.6	46.1	40.2	41.1

Bulk Density: The bulk density of the soil for the 20 to 115 cm depth interval in the drip field ranged from 1.57 to 1.75 g cm^{-3}. For the spray field, the bulk density in the 7 to 210 cm depth interval varied between 1.57 to 1.83 g cm^{-3}. In general, these values appear to be high for the type of clayey soils found at the site, indicating relatively low total porosity for all horizons.

Saturated Hydraulic Conductivity: The maximum depth of sampling from which intact cores were obtained for laboratory determination of K_{sat} was 115 cm for the drip field and 210 cm for the spray field. The in situ and laboratory-determined K_{sat} for the spray and drip fields are given in Fig. 30. For both fields, K_{sat} decreased with depth and the laboratory-determined values were less than 0.1 cm d^{-1} for the samples collected from the Bt horizons. In fact, for some of the intact cores, no measurable amount of water could pass through them during K_{sat} measurement. In the upper 30 cm of the soil, laboratory K_{sat} values were less than 5 cm d^{-1} but the in situ values ranged from 1.5 to approximately 11 cm d^{-1}. Overall, the results for the top 30 cm corresponded well with the more sandy texture of the A and E horizons. With few exceptions, K_{sat} of all the cores below 80 cm was less than 0.1 cm d^{-1}. These low K_{sat} values corresponded well with the very sticky, very plastic, clayey Bt1 horizons within the spray and drip fields. Generally, the agreement between the laboratory-determined and in situ K_{sat} values are fairly good, and measured values by both techniques indicate the restrictive nature of the soils at this site at depths greater than 50 cm. These results are consistent with the other soil properties that limit the use of subsurface wastewater disposal systems in the types of soil found at the site.

Soil Water Characteristic: The average soil water retention values for the entire profile of the spray field and drip field at saturation (i.e., zero soil water pressure head) were 0.406 and 0.432 m^3 m^{-3}, respectively (Table 5). The individual soil water content values at saturation for different depth intervals ranged between 0.360 to 0.511 m^3 m^{-3} for the drip system, and between 0.292 and 0.476 m^3 m^{-3} for the spray field. Although individual average values under each soil water pressure head were different for the two systems, the amounts of water drained between saturation and -400 cm, and between saturation and -15,000 cm were fairly close for the two systems.

Drip System

Ponding in the Trenches: No gravel was used around the lateral lines of the drip system. Therefore, any ponding in the observation wells installed at different places in the original trenches where the drip lines were buried indicates saturated conditions. Except for the dry period during the summer months, ponding was observed at different locations within all five trenches. Figure 31 presents the level of liquid in the observation wells installed at the beginning, middle, and end of lateral 2 with a loading rate of 0.05 gal ft^{-2} d^{-1}, and lateral 5 with a loading rate of 0.1 gal ft^{-2} d^{-1}. West (1994) presents the data for all five lateral lines. For lines 1 to 4 (see West, 1994), the greatest amount of ponding was at the middle position, but for the fifth line ponding was the highest at the end of the lateral line. In fact, ponding was observed at this position for most of the times during the study (see Fig. 31B).

We should note again that the trenches were not installed on contour, no leveling or cleaning of the bottom of the trenches was performed, and the laterals were not installed with a known or

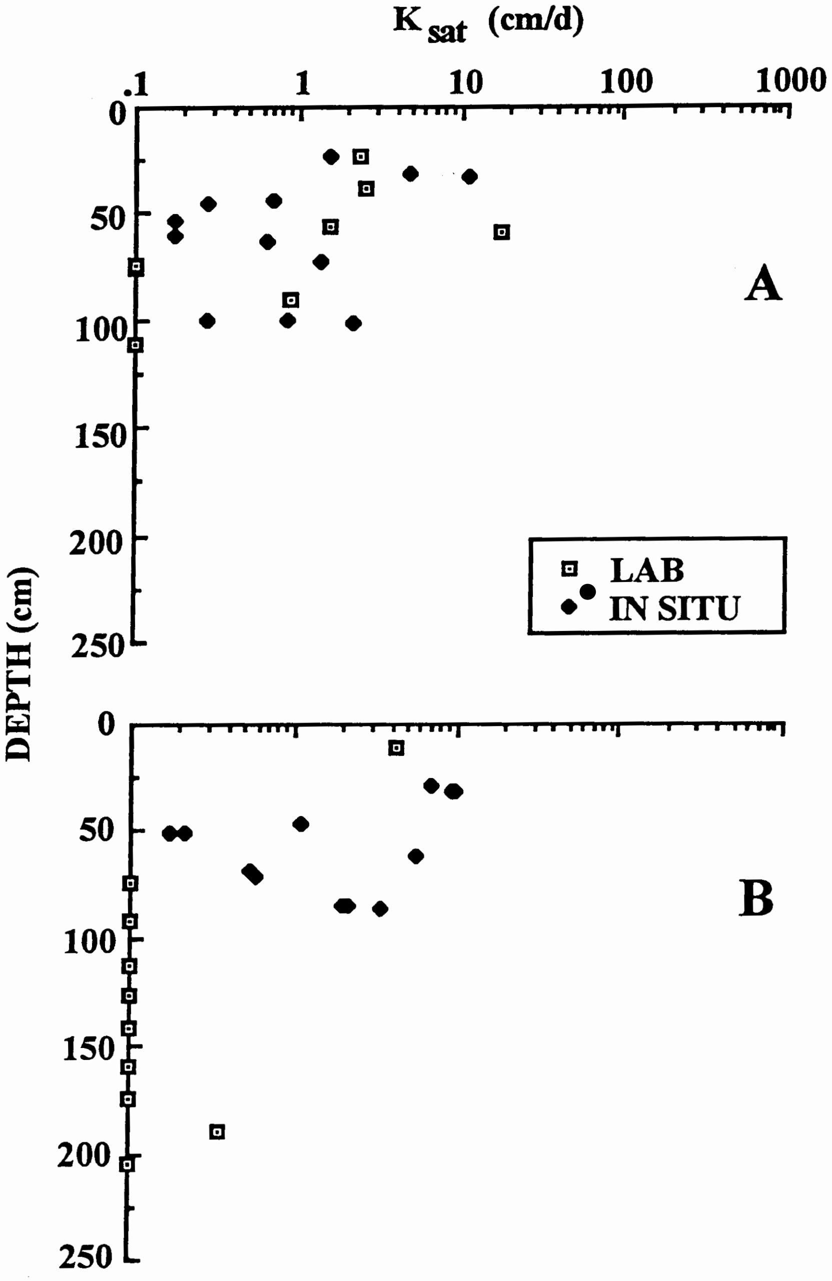

Figure 30. In situ and laboratory-determined K_{sat} for (A) the spray field and (B) the drip field at the Chatham County site.

Table 5. Mean, standard deviation (s.d.), and number of observations (n) for soil water retention under various soil water pressure heads for the spray irrigation and drip fields at the Chatham County site.

FIELD		SOIL WATER PRESSURE HEAD, cm										
		0	-25	-50	-100	-150	-200	-300	-400	-1,000	-5,000	-15,000
		m³ m⁻³										
Drip	Mean	0.432	0.419	0.420	0.410	0.401	0.394	0.387	0.379	0.355	0.307	0.260
	s.d.	0.044	0.043	0.02	0.057	0.061	0.062	0.064	0.066	0.094	0.089	0.080
	n	8	8	8	8	8	8	8	7	7	8	8
Spray	Mean	0.406	0.392	0.398	0.390	0.384	0.377	0.367	0.358	0.360	0.276	0.221
	s.d.	0.054	0.068	0.061	0.065	0.068	0.071	0.073	0.075	0.066	0.108	0.086
	n	10	10	10	10	10	10	10	7	10	10	10

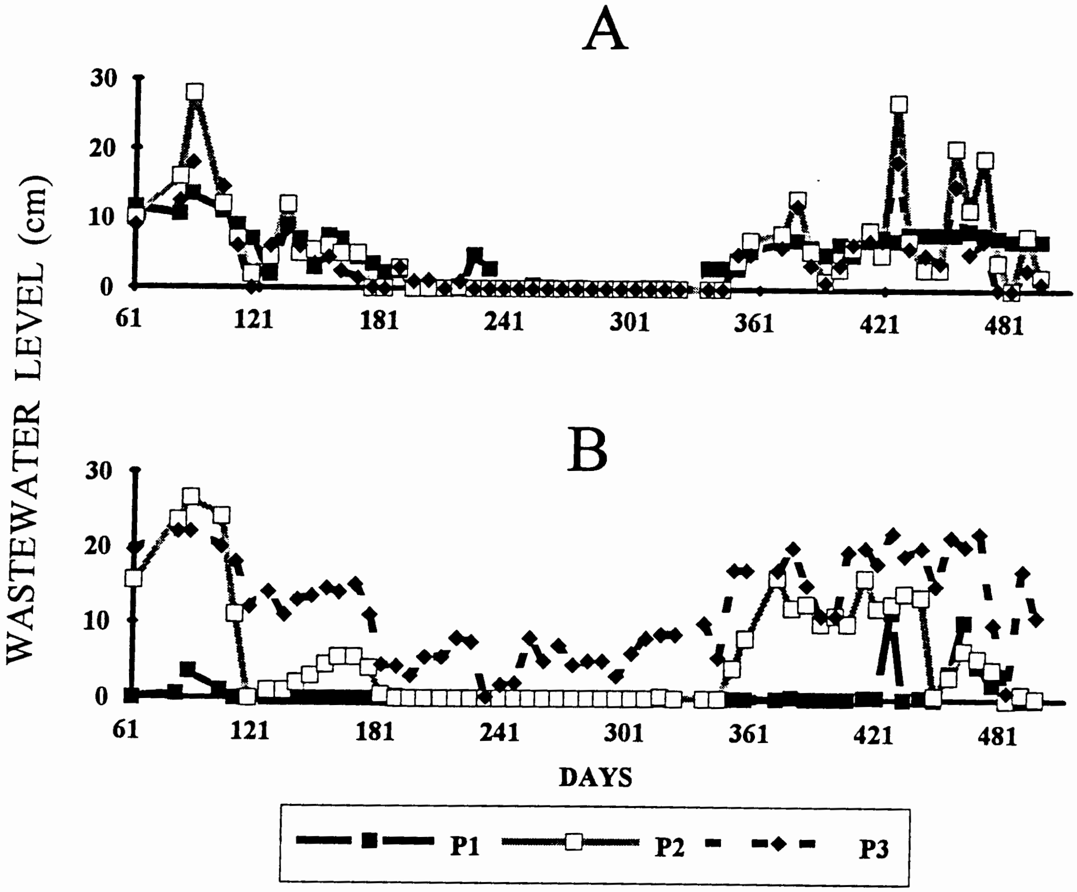

Figure 31. Wastewater ponding at the beginning (P1), middle (P2), and end (P3) of the trenches of the drip system for (A) the second drip line loaded at 0.05 gal ft^{-2} d^{-1} and (B) the fifth drip line loaded at 0.1 gal ft^{-2} d^{-1} at the Chatham County site.

uniform slope. In addition, the trenches were dug into the Bt horizon and the sidewalls of the trenches were smeared during the installation process. As a result, infiltration of water or wastewater from the trenches into the surrounding soil was substantially reduced. Although lateral movement of rainwater infiltrating the relatively permeable surface soil above the drainfield area into the trenches and lack of uniform wastewater distribution over the drainfield are logical reasons for some of the ponding, the presence of low spots in the trenches and redistribution of wastewater inside the trenches are other factors that could have resulted in high levels of ponding at some locations. We should point that, although ponding was observed for an extended period of time in the trenches, no surfacing of wastewater was observed at any location within the drainfield area. This indicates that additional studies are needed to assess the potential of shallow installation of drip lines in the relatively permeable surface horizons underlain by slowly permeable Bt materials in the Triassic Basin.

Soil Water Pressure Head: Above the drip system, the soil water pressure head values for 20, and 60 cm depths above the drip system were negative for an extended period of time during the dry periods (Fig. 32A). At 40 cm depth, soil water pressure head did not change significantly with the season. The 20 cm depth perhaps represented the sandy loam materials at the surface, and the 60 cm depth was well within the Bt2 horizon. The 40 cm depth, however, represented the clayey Bt1 horizon and remained wet for the duration of the study. Below the drip field, the soil water pressure head was near zero and remained relatively unchanged at 60 cm depth (Fig. 32B). Because this position was near a shallow ditch (a natural drain), the upper 40 cm of the soil had a chance to drain, whereas at 60 cm depth, the soil remained wet.

The soil water pressure head values for the bottom of the trench and 20 cm below the trench for lines 2 and 5 are shown in Fig. 33, and the results for other lines are given by West (1994). The soil water pressure head at both depths for line 1 (see West, 1994) remained negative during the dry periods, whereas for lines 3 and 4 the soil water pressure head remained near or above zero for most of the times. These results corresponded well with the wastewater ponding within trenches 1, 3, and 4, respectively. The soil water pressure head within trenches 2 and 5 also corresponded fairly well with the trench ponding data. For line 2, the soil water pressure head at 20 cm below the bottom of the trench was positive even when there was no ponding in the trench and the soil water pressure head at the bottom of the trench was ≤ zero. This indicates that the Bt horizon remained saturated or near saturated for the entire study period, which is consistent with the results of measurement of soil water content adjacent to the trench (the results will be presented later). For line 5, the soil water pressure head values at the bottom of the trench and at 20 cm below the bottom of the trench were at or above zero for most of the time, but showed unsaturated conditions during the dry periods. These results are consistent with the trench ponding and soil water content data that will be presented later.

The soil water pressure head values for three depths near the trenches 2 and 5 are shown in Fig. 34. Similar to the results for the soil water pressure heads below the trench bottom for line 2, the soil water pressure head values at the 40 and 60 cm depths indicated saturated conditions for most of the study period, whereas soil water pressure head at the 20 cm depth was mostly negative. For line 5, the soil water pressure head values for all three depths (Fig. 34B) were at or above zero during the wet periods, but were substantially negative (as low as -800 cm) for a

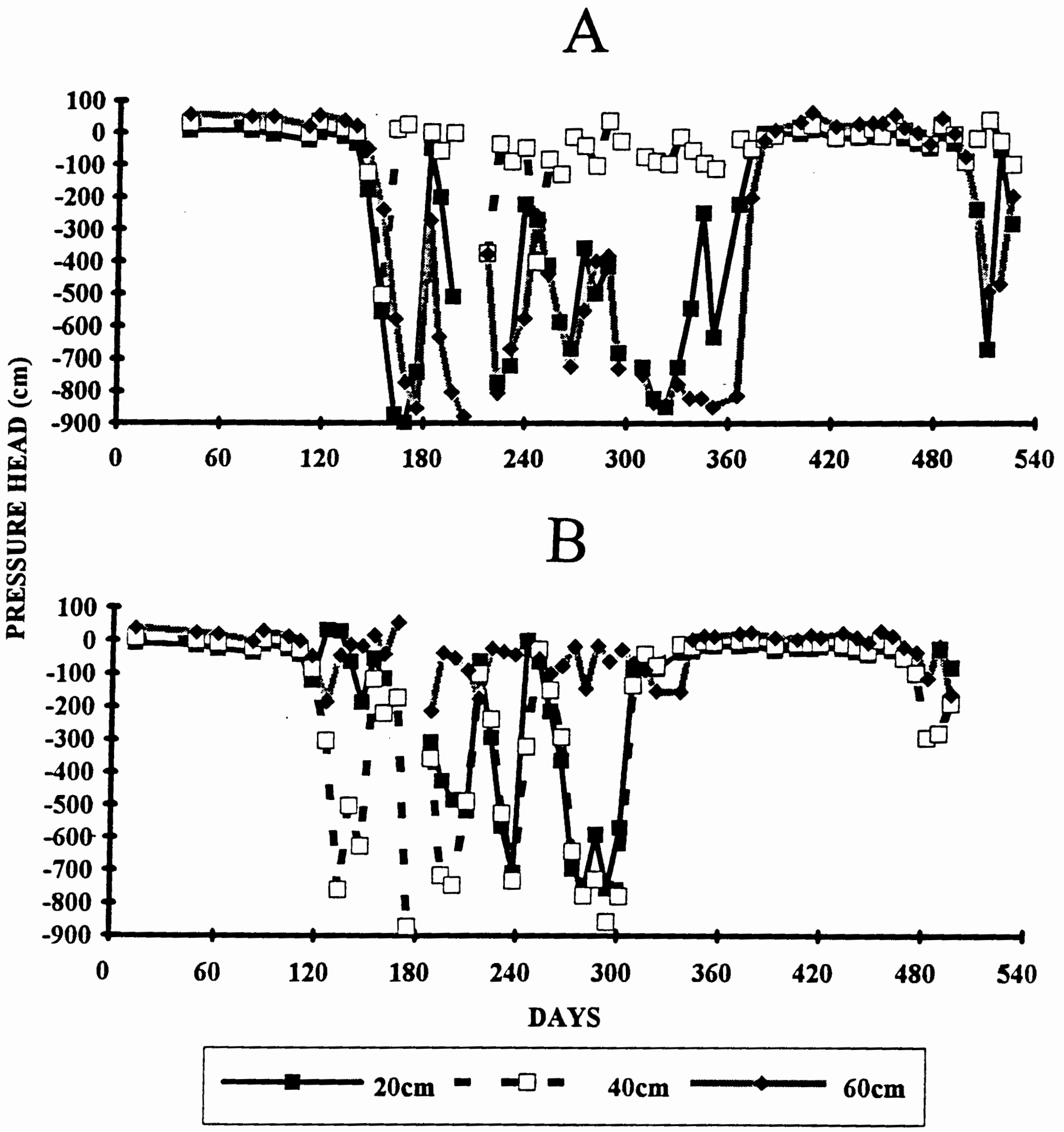

Figure 32. Background soil water pressure head values for three depths for (A) between the drip system and spray irrigation field and (B) below the drip system near the natural drainage at the Chatham County site.

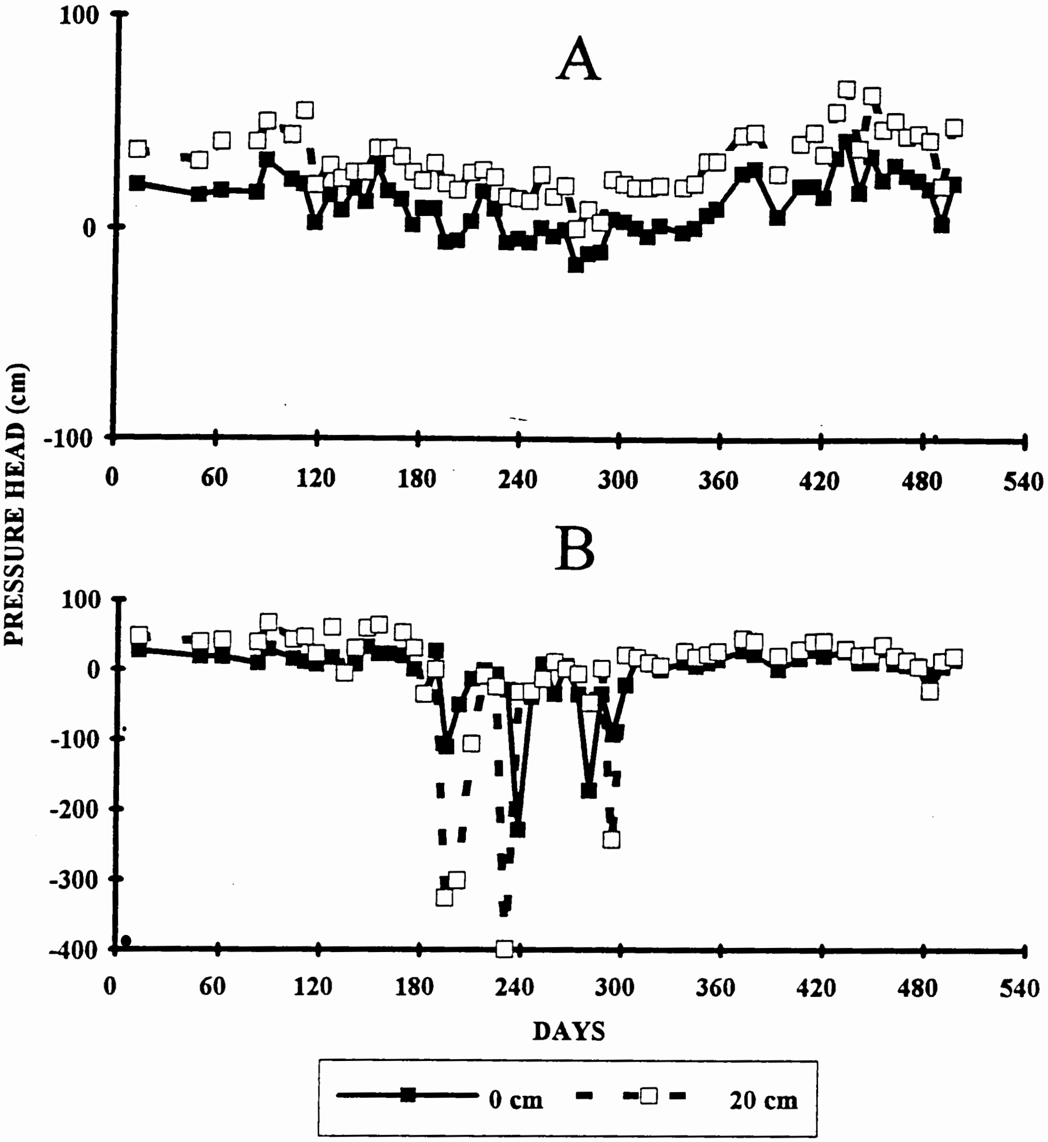

Figure 33. Soil water pressure head values for two depths below the trench bottom for (A) drip line 2 loaded at 0.05 gal ft^{-2} d^{-1} and (B) drip line 5 loaded at 0.1 gal ft^{-2} d^{-1}.

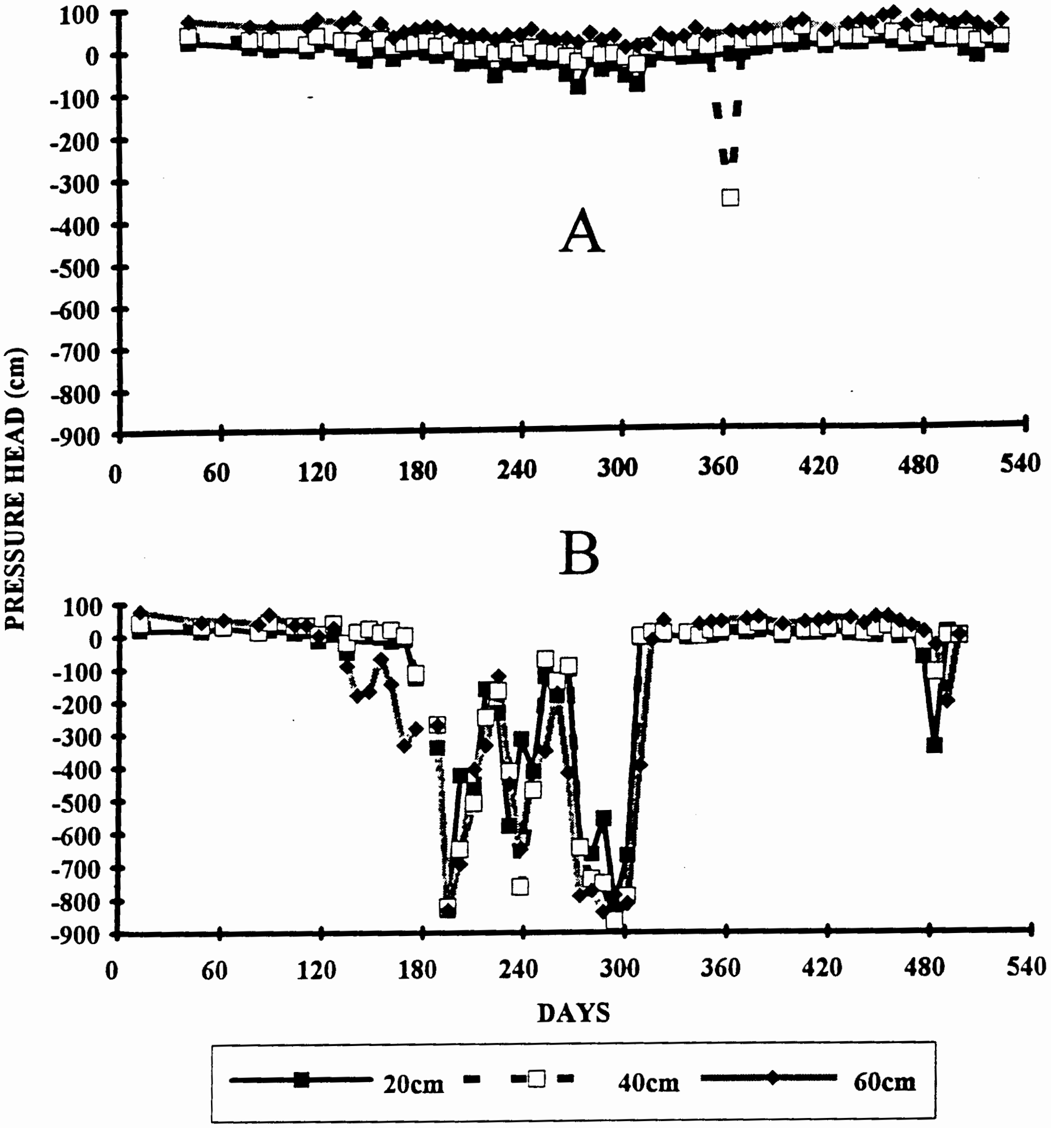

Figure 34. Soil water pressure head values for three depths adjacent to (A) the drip line 2 loaded at 0.05 gal ft^{-2} d^{-1} and (B) the drip line 5 loaded at 0.1 gal ft^{-2} d^{-1}.

majority of the dry period during the summer and fall months. For the other lines, the results show the restrictive nature of the Bt horizon where the upper part of the Bt horizon became wet and prevented downward movement of water to lower layers.

Soil Water Content: The soil water content from the soil surface to 60 cm depth was measured by TDR at 15 cm distance from the trench wall at the beginning (next to the manifold) and at the end of each drip line. West (1994) presented the calculated soil water content for four depth intervals for all locations. Due to the similarity of the results for the three lines receiving 0.05 gal ft^{-2} d^{-1} (i.e., lines 1 to 3) and the two lines receiving 0.1 gal ft^{-2} d^{-1} (lines 4 and 5) only the results for lines 2 and 5 will be presented here.

The soil water content in the upper 15 cm of the soil at the beginning and end of line 2 demonstrated seasonal variations (Fig. 35). During the summer months, when evapotranspiration exceeded precipitation, the soil water content near the end of the line was less than 0.1 m^3 m^{-3}. At deeper depth intervals, the soil water content remained between 0.35 and 0.45 m^3 m^{-3} for most of the times. These results agree with the soil profile in the upper part of the drainfield and the soil water retention data (data for individual lines are not given). The sandy loam A horizon in the upper part of the profile drained easily and did not have a high water holding capacity. On the other hand, the Bt1 horizon below the A horizon had a clayey texture and was very sticky and very plastic. This horizon did not drain easily and remained relatively wet for the entire year. At the lower part of the drainfield, near a shallow natural ditch, the A and E horizons were 30 cm thick. The thicker A and E horizons combined with the natural drainage that occurred near the ditch (i.e., edge effect) resulted in more drying in the upper layers (0 to 15 and 15 to 30 depth intervals) and higher temporal variability in the soil water content at all depths (Fig. 36). Also, the trenches at the bottom of the drainfield were shallower than the trenches in the upper part of the field. In the upper part of the field, wastewater was directly placed in the Bt1 horizon, whereas in the lower part of the field wastewater was applied to the sandy loam A and E horizons. As a result, the drip system appeared to function better in the lower part than the upper part of the field even though the loading rate at the lower part was twice the loading rate at the upper part of the field.

Spray Irrigation System

Wastewater Distribution Uniformity: Initially, the distribution of wastewater over the spray irrigation field was evaluated for a 7-day period. As a result, rainfall during the period was also collected in the catch cans placed on a regular pattern over a portion of the spray field covering three neighboring nozzles (see Fig. 8). Later, only the distribution of wastewater over the spray field was determined by measuring the amount of wastewater collected in the catch cans for a given dosing cycle. Figures 37 and 38 present the distribution of wastewater and precipitation for a 7-day period during January and June, 1993. For January, 1993 (Fig. 37), the vegetative cover of the spray field had little foliage and the distribution of the combined precipitation and wastewater was less uniform than during June (see Fig. 38) when foliage was a major factor. Comparing the two figures, a high level of rain and wastewater was collected at a point near a tree as shown on the upper left hand side of the area. The total amount of rain water and

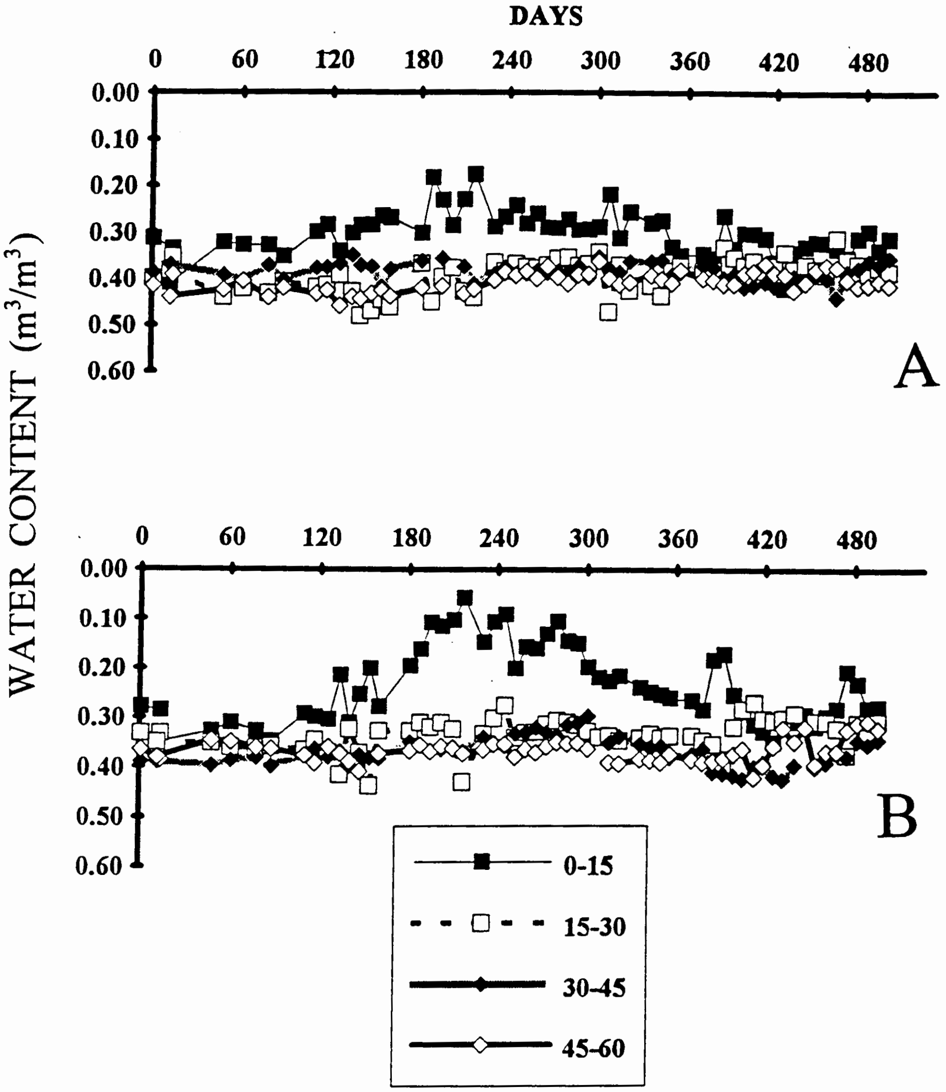

Figure 35. Soil water content for four depth intervals at 15 cm from the drip line for (A) the beginning and (B) the end of line 2 loaded at 0.05 gal ft^{-2} d^{-1}.

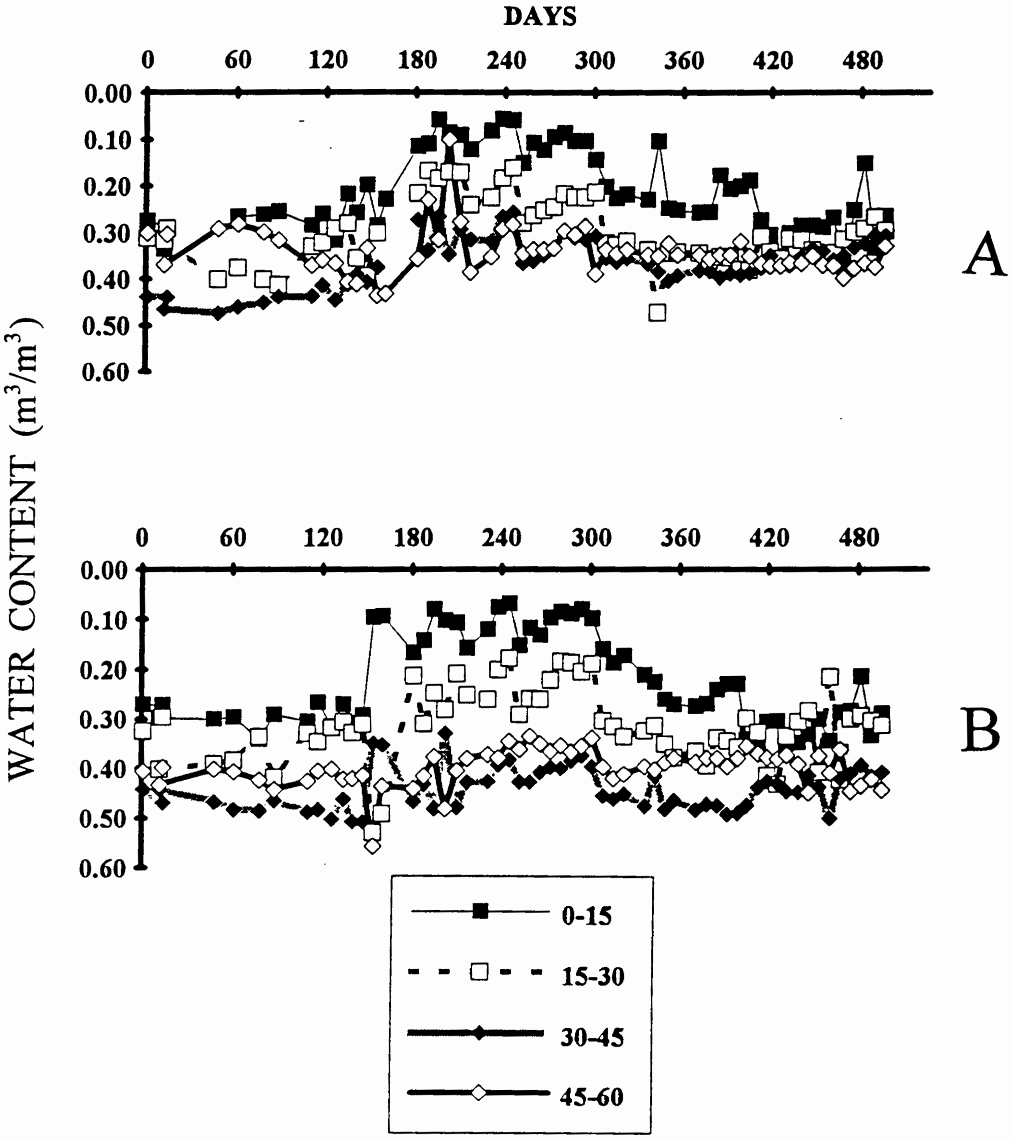

Figure 36. Soil water content for four depth intervals at 15 cm from the drip line for (A) the beginning and (B) the end of line 5 loaded at 0.1 gal ft^{-2} d^{-1}.

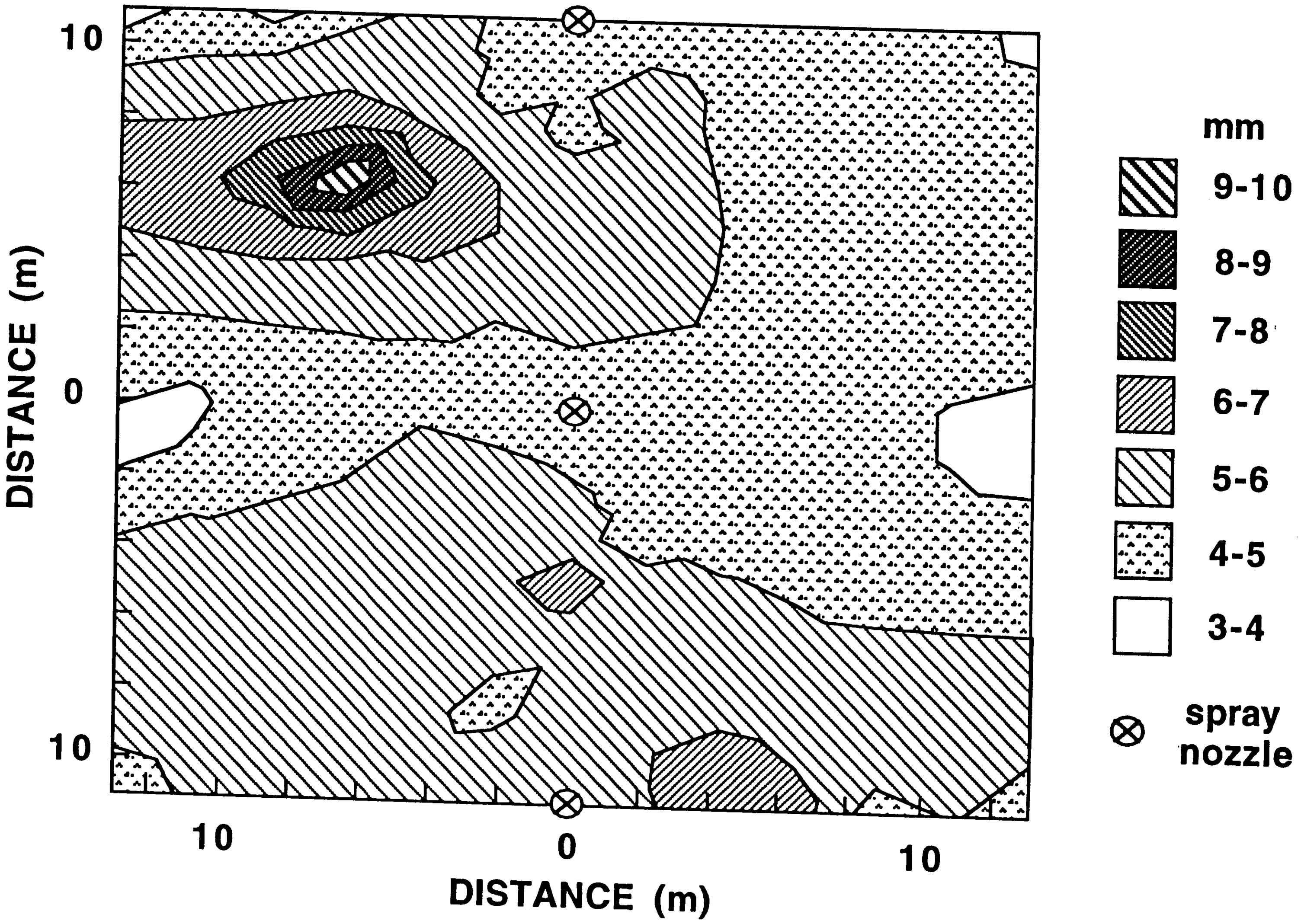

Figure 37. Distribution of wastewater applied by the spray irrigation and rainfall over part of the spray field for the week of January 26, 1993, at the Chatham County site.

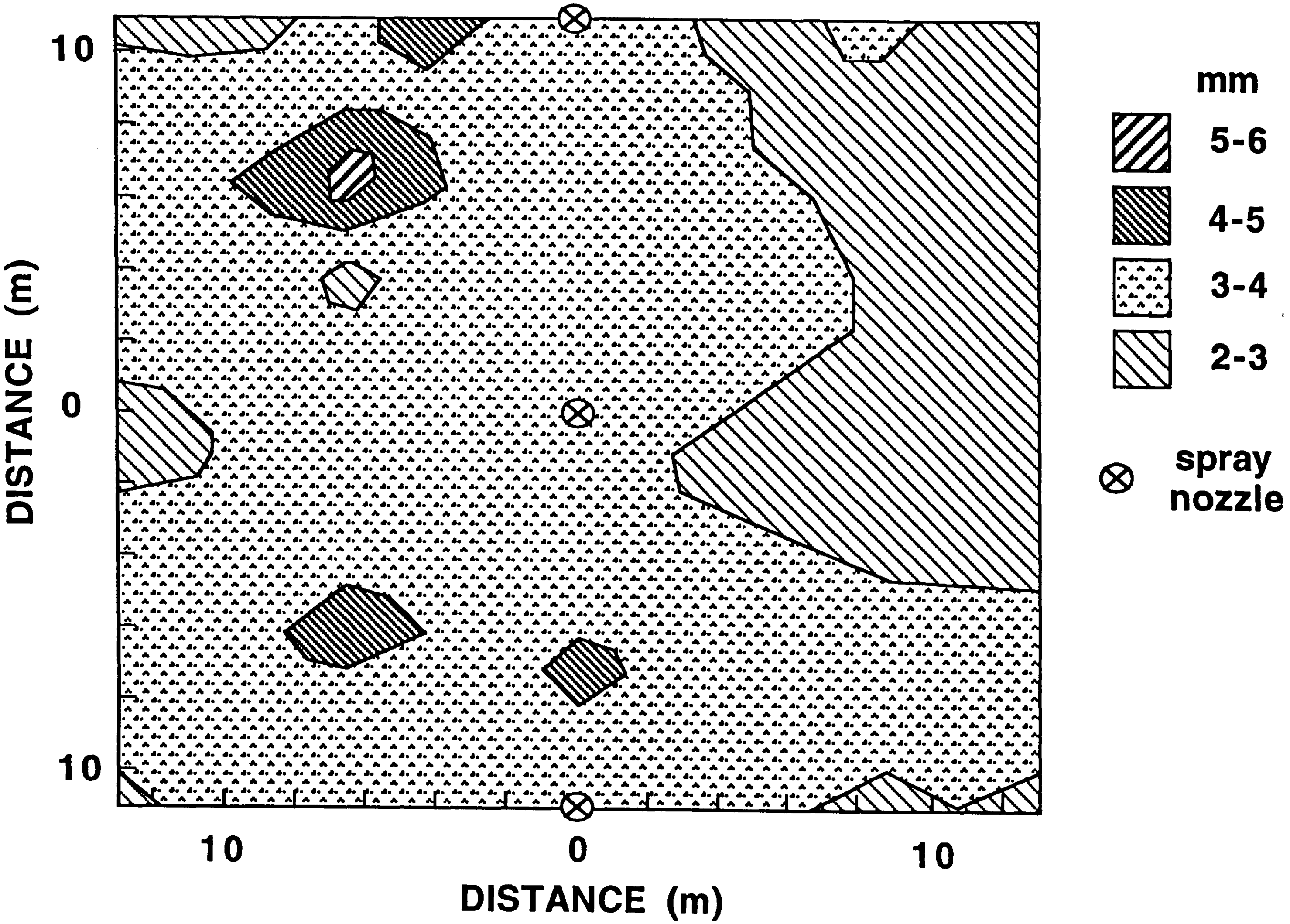

Figure 38. Distribution of wastewater applied by the spray irrigation and rainfall over part of the spray field for the week of June 17, 1993, at the Chatham County site.

wastewater collected in the upper right hand side of the area was substantially less than the left and lower parts of the area. We believe the lower amount is due to the type of vegetative cover as well as to the prevailing wind causing some drift across the field.

The amount of wastewater collected during a dosing event in March, 1994, when foliation was not a factor (Fig. 39), and in May, 1994, when foliation was becoming thick (Fig. 40), show the pattern of wastewater distribution around individual spray heads. For March, wastewater was distributed over a larger area, whereas in May, some parts of the field received little or no wastewater. This is most likely due to the foliage intercepting the applied wastewater. Similar to the previous measurements, the upper right hand side of the field received the least amount of wastewater and the amount of wastewater applied to the field was highest in the middle of the field where areas covered by individual spray heads overlapped. Overall, the distribution patterns of wastewater applied by the sprinkler system and natural precipitation over the land surface were not uniform in the presence or absence of foliage. Considering the low loading rates for most single family wastewater disposal systems, we believe the lack of a completely uniform distribution pattern for the spray irrigation system will have a minimal effect on the overall distribution of water from combined precipitation and wastewater over the land surface.

Runoff Analysis: Figure 41 presents the pH, electrical conductivity (EC), nitrate and ammonium concentrations of the runoff samples from the spray field with time. We should note that due to low wastewater application rate runoff samples could only be collected after relatively high rainfall events. Except for one sampling event, the pH of the runoff was slightly acidic. Considering the pH of rain and the pH of the type of the soils found at this site, the applied wastewater will have little effect on the pH of the soil. The EC of the runoff samples was generally low (< 0.16 dS m^{-1} or mmhos cm^{-1}) and should pose no salinity hazard in these soils. In the runoff, nitrogen was mostly in the form of nitrate with small amount in the form of ammonium. Lack of ammonium in the runoff is due to the pretreatment of the septic tank effluent by an aerobic unit and the surface application of wastewater. Nitrate concentration exceeded 10 mg L^{-1} only one time during our monitoring. Due to the lack of data on the nitrogen concentration in wastewater entering the system daily, the cause of occasional relatively high nitrate concentration in the runoff cannot be explained.

Wastewater Analysis: The results for the BOD_5, COD, and fecal coliform of wastewater samples from the septic tank, the ATU (after treatment and before chlorination), the pump tank (after chlorination), and one location within the drip system are given in Table 6. As shown in the table, substantial reduction in BOD_5, COD, and fecal coliform had occurred during each stage of the system. It appears that the chlorination in this wastewater management system has had little effect on the fecal coliform, but the number of bacteria is substantially reduced after wastewater is applied to the drip system. The reduction in the number of bacteria in the drip field could be due to the natural die off of the bacteria and/or dilution resulting from mixing of wastewater with infiltrating rain water. The chlorinator in this system consisted of a perforated PVC cylinder that contained chlorine tablets. This cylinder was placed, through a port, in the line connecting the ATU and the pump tank. It is assumed that wastewater passing by the cylinder, or going through the perforations, comes in contact with chlorine and become disinfected. However, we believe this type of chlorinator is not an effective mechanism for disinfection due to the very short time

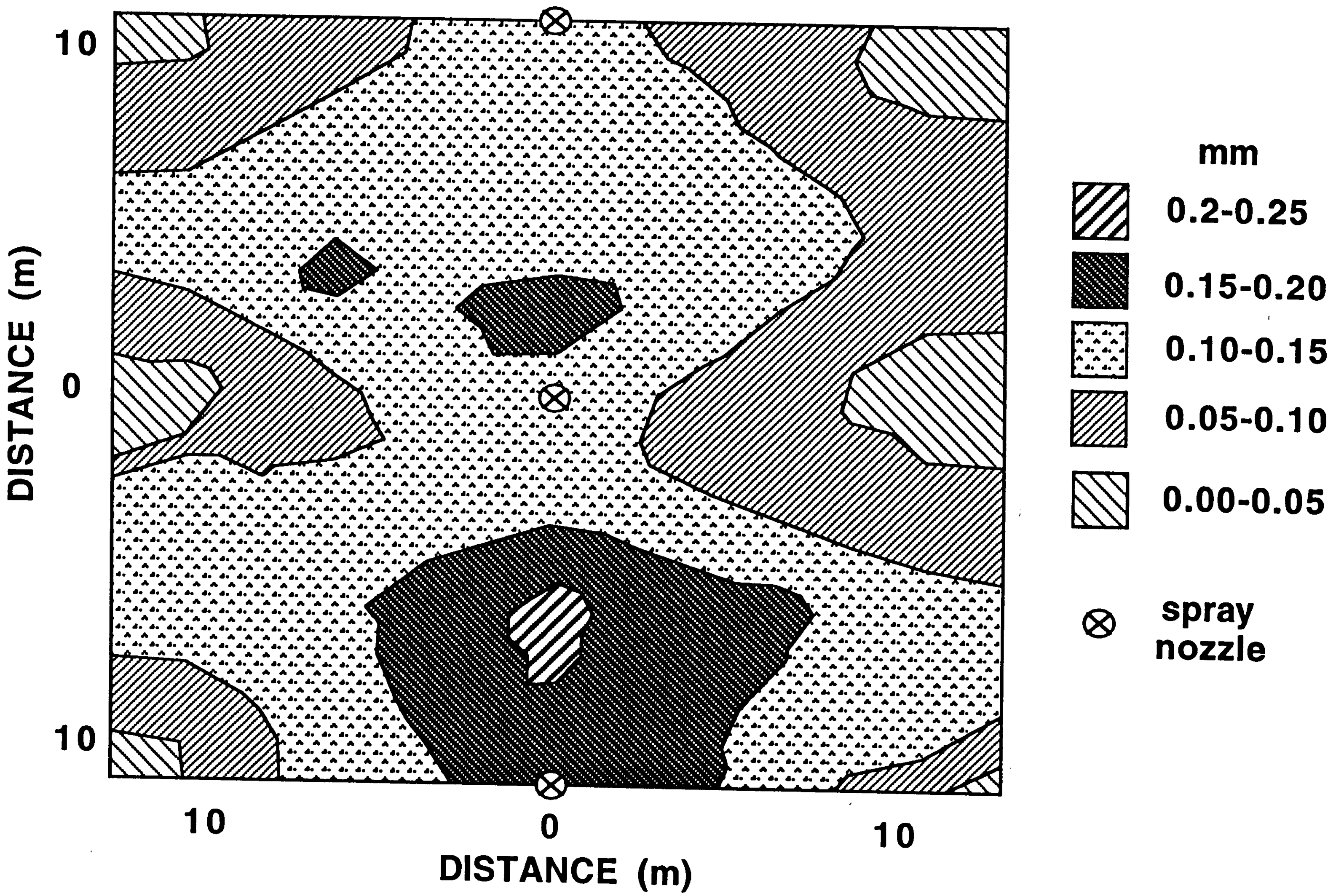

Figure 39. Distribution of wastewater applied by the spray irrigation over part of the spray field for one dosing event on March 30, 1994, at the Chatham County site.

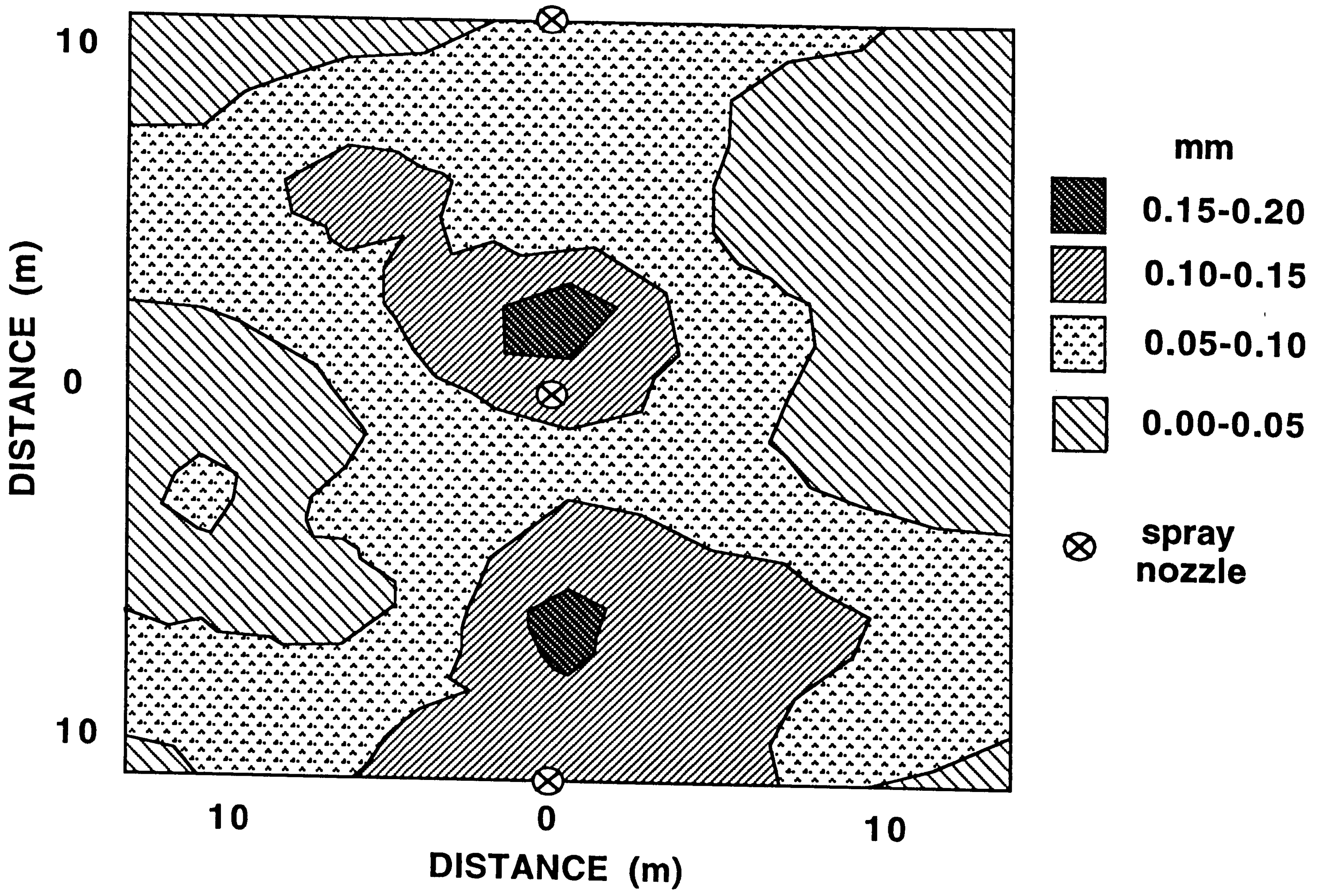

Figure 40. Distribution of wastewater applied by the spray irrigation over part of the spray field for one dosing event on May 11, 1994, at the Chatham County site.

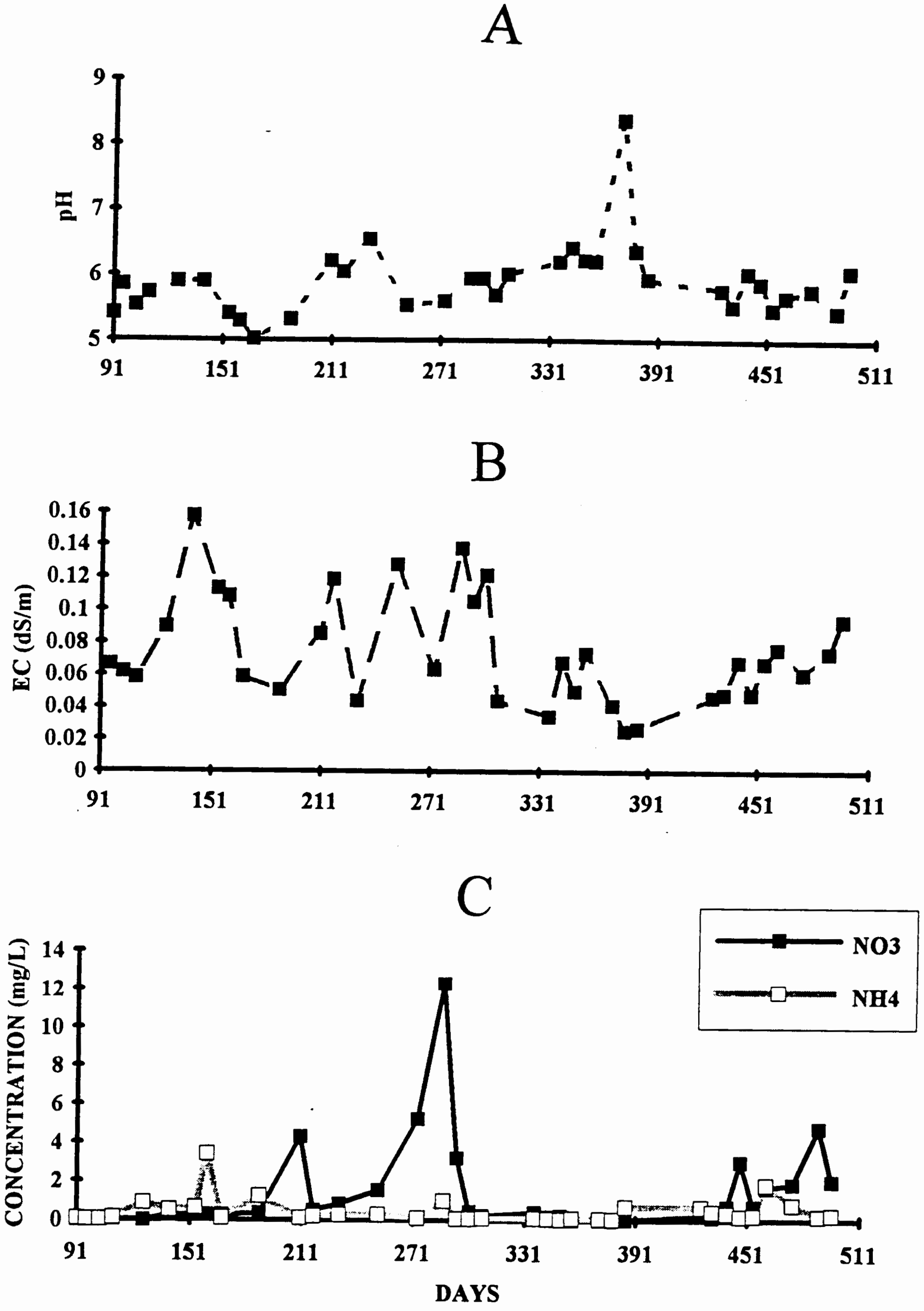

Figure 41. Values for (A) pH, (B) electrical conductivity (EC), and (C) nitrate and ammonium concentrations for runoff collected from the spray field at the Chatham County site.

Table 6. Selected characteristics of wastewater collected from various components of the septic system at the Chatham County site.

SOURCE OF SAMPLE	BOD_5	COD	FECAL COLIFORM
	mg L^{-1}		count per 100 mL
Septic Tank (before aerobic treatment)	260 - 1,370	261 - 2,020	34,700 - 56,300
ATU (before chlorination)	59 - 1,700	153 - 6,940	4,100 - 6,300
Pump Tank (after chlorination)	22 - 982	57 - 3,550	477 - 32,000
Drip Field (ponded trench)	24 - 81	105 - 198	100 - 593

period during which wastewater can contact the chlorine tablets. The levels of BOD_5 and COD of the wastewater in the pump tank and the drip field also were higher than the comparable values at the Wake County site. Better management of the ATU, and proper disinfection can perhaps lower the BOD_5, COD, and fecal coliform concentration of the septic tank effluent more effectively.

Distribution of Water from Drip Systems

As indicated earlier, the drip lines used at the Wake County site (hereafter referred to as System I) contained one orifice at 60-cm interval, whereas the drip lines used at the Chatham County site (hereafter referred to as System II) contained two orifices (on the opposite sides) at 60-cm interval. For all three installation scenarios (level, nearly level with slight variation in elevation of orifices, and sloping at 1.25%), the rate of water flow from individual orifices of System I in the middle of a dosing period (i.e., when the system was fully pressurized or final flow rate) was approximately half the corresponding final flow rate from orifices of System II (Table 7). The variability for the final flow rate among the orifices for both systems, however, was fairly low. The final orifice flow rate for System I under 35 m (50 psi) pressure head corresponded fairly well with the respective design flow rates of 38 cm^3 min^{-1} (0.6 gal h^{-1}), but for System II the flow rate under 14 m (20 psi) pressure head was approximately 25% higher than the design flow rate of 65 cm^3 min^{-1} (1 gal h^{-1}). Based on the measured volume of water discharged from each orifice during three measurement periods, a total flow volume from each orifice was calculated for a 16-min pumping period for System I and for an 8-min pumping period for System II. For this

Table 7. Mean and standard deviation (in parentheses) for the amount of flow from orifices of the two drip systems for three different installations. The initial period is for the first minute of operation, the middle period is the final flow rate when the system is fully pressurized (units cm^3 min^{-1}), and the drainage period is for the amount of water drained from the orifices after stopping water application.

Line	Period	Zero Slope	Field Installation*	1.25% Slope
		cm^3		
SYSTEM I				
1	initial	39.2 (4.6)	52.4 (3.36)	41.0 (4.7)
	middle	40.8 (0.8)	39.4 (1.2)	38.3 (0.5)
	drainage	55.3 (42.0)	75.9 (58.5)	113.5 (103.1)
4	initial	23.9 (4.3)	45.4 (4.9)	32.7 (1.8)
	middle	39.4 (1.3)	39.6 (1.2)	38.2 (1.06)
	drainage	60.7 (42.5)	74.0 (46.4)	96.2 (93.9)
7	initial	-1.8 (4.3)	26.2 (2.7)	13.8 (6.9)
	middle	40.1 (0.9)	40.4 (1.0)	38.2 (0.8)
	drainage	55.2 (36.6)	44.4 (40.4)	85.7 (84.0)
SYSTEM II				
1	initial	83.5 (6.5)	38.1 (12.3)	4.2 (7.2)
	middle	81.8 (2.2)	83.2 (2.1)	81.8 (3.4)
	drainage	25.2 (17.9)	58.3 (43.3)	110.8 (63.4)
4	initial	80.2 (4.7)	32.9 (3.0)	-1.3 (3.1)
	middle	82.3 (2.0)	82.4 (1.7)	82.4 (1.7)
	drainage	21.1 (14.6)	49.0 (38.3)	96.5 (58.2)
7	initial	81.2 (3.3)	33.5 (1.6)	-0.7 (2.7)
	middle	84.2 (1.5)	84.2 (1.9)	83.5 (1.8)
	drainage	14.1 (11.5)	58.2 (52.7)	108.0 (54.9)

* Refers to the installation when the elevations of orifices were varied along the drip lines.

purpose, the combined discharge values during the first minute and during the drainage period were added to the total flow calculated for 15 or 7 min using the final flow rates for Systems I and II, respectively. Figures 42 and 43 present the calculated total volume of water discharged from each orifice along three lines of Systems I and II, respectively, for a level (zero slope), a nearly level (field installation, variations in the elevation along the drip lines was within 5 cm), and a drip system installed with a uniform slope of 1.25% along the drip lines. For both systems, the averages for the flow volume from the respective orifices during a dosing period were approximately the same. Due to the differences in the layout and design of the two systems (see Fig. 9), for zero slope installation, the calculated total flow from individual orifices in System I (16-min dosing period) was slightly more variable than the flow volume from orifices of System II (8-min dosing period), but the coefficient of variation (CV= 100 x standard deviation/mean) values for both systems were fairly low (< 7%). The variability of total flow from orifices increased when there were differences in the elevations of the orifices along individual lines (simulating field installation on a flat land). The CV values for the total flow for the three drip lines when they were not completely level were between 6.4 and 9.6% for System I and were between 6.7 and 8.7% for System II. When the systems were installed with a continuous slope along their lines, the lower points of each system discharged the highest volume of water from orifices due to a greater amount of free drainage (Figs. 42C and 43C). Among the three installations, the variability of flow volumes along each line for sloping system was the highest (CV values 8.5 to 14.9%).

For each system, no significant change in the final flow rate values was observed when the laterals were not completely level as compared to the case when the lines were level (see the middle periods in Table 7). For a completely level System I, with direct feeding of water from one end of the system, the flow rate for the first line reached the final flow rate within a short time, whereas for the last line (7th line in our experiment) the flow rate did not reach the final flow rate within the first minute of operation. When the system was installed on a uniform slope of 1.25%, the volume of drainage was substantially higher than when the system was on a level surface due to water moving out of the lower holes. When System II was installed level, the volume of flow during the first minute of operation reached the final flow rate rather quickly, and the amount of drainage was the lowest of all installations. These results were perhaps due to the fact that both ends of each drip line were connected to two manifolds equipped with air release vents (see Fig. 9), and that its orifice design requires water to flow around the inside perimeter of the pipe before exiting the orifice. It appears that drainage stops when air enters the orifice mechanism. When the system is repressurized, the drip lines are relatively full, and air can escape from the air vents easily. When System II was installed nearly level, the volume of drainage from some orifices at the lower parts of the lines increased substantially. In System II, for a uniform slope of 1.25%, the final flow rate did not change, but the volume of drainage was substantially higher while the volume of flow during the initial period of pumping was substantially lower than the corresponding values for a level installation. Although the major amount of water flow from the orifices is during the time when a drip system is fully pressurized, the effects of non-uniformity of flow from the orifices during the initial and final (drainage) periods on the overall uniformity of wastewater application may be significant. This is particularly true when the system is dosed frequently.

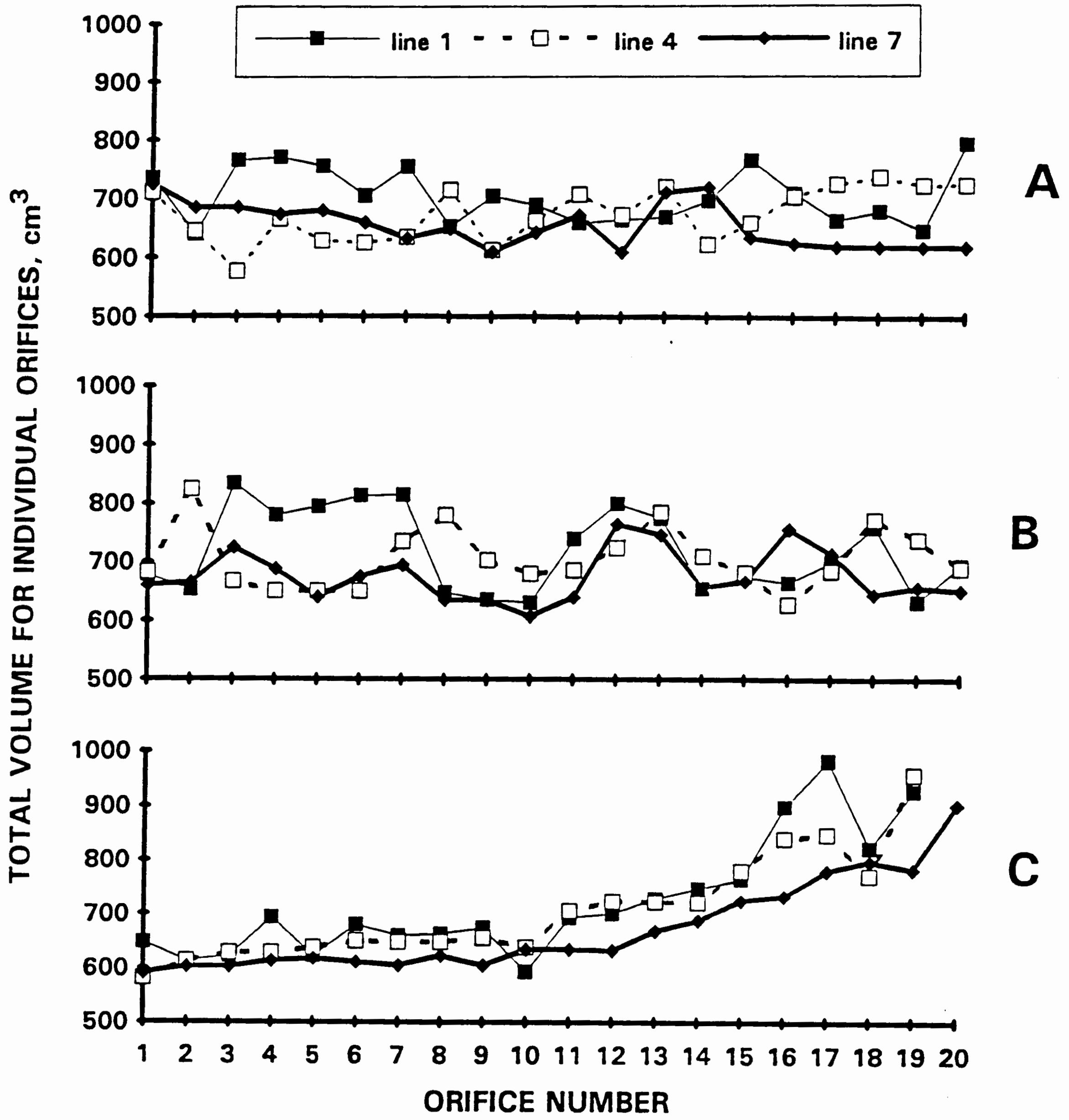

Figure 42. Total volume of water flowing out of individual orifices on three drip lines for drip System I for (A) a level installation, (B) a nearly level simulating a field installation, and (C) continuous uniform slope of 1.25% along the lines. The dosing period was 16 min, with a design loading rate of approximately 40 cm^3 min^{-1} for each orifice.

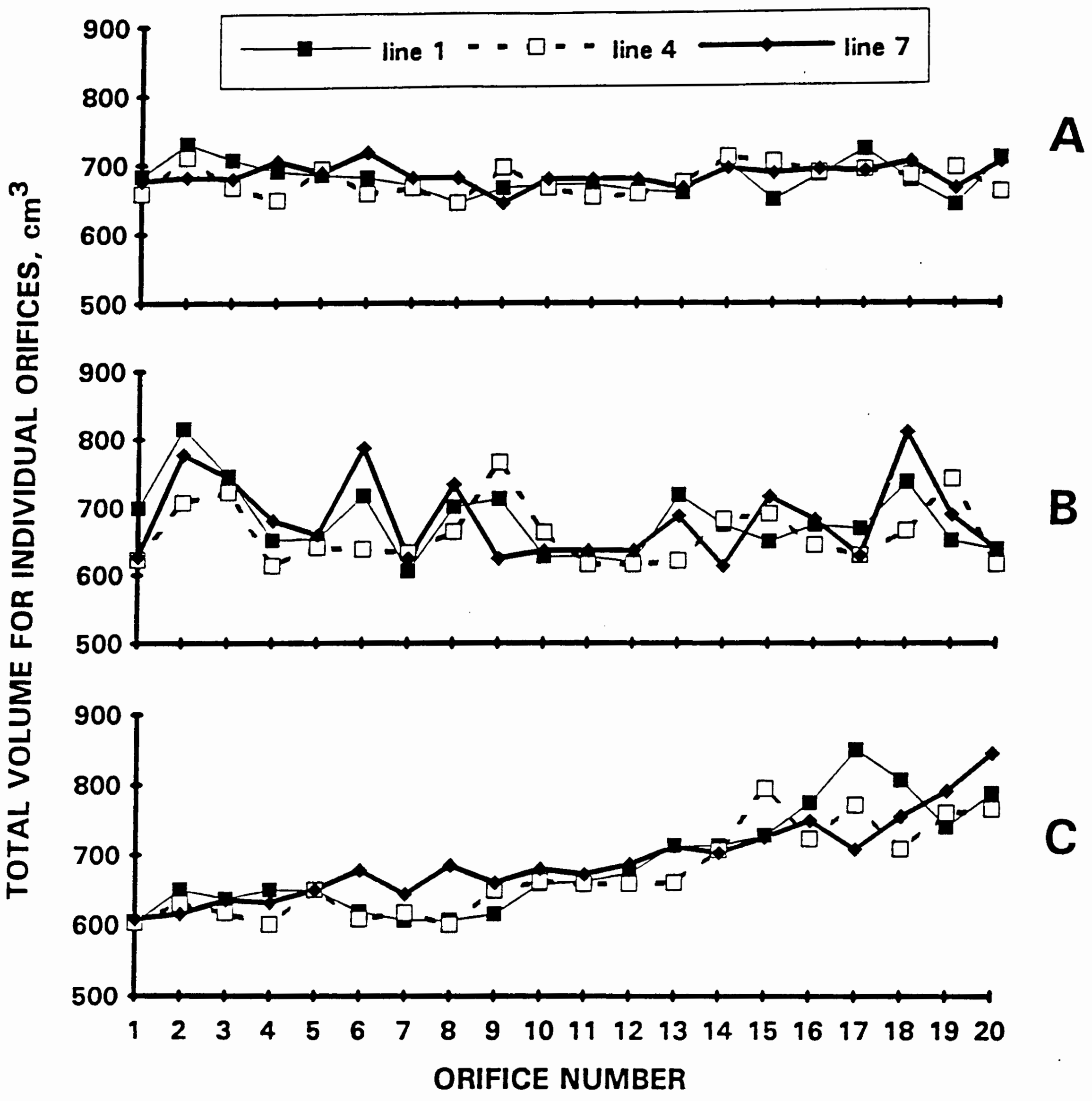

Fig. 43. Total volume of water flowing out of individual orifices on three drip lines for drip System II for (A) a level installation, (B) a nearly level simulating a field installation, and (C) a continuous uniform slope of 1.25% along the lines. The dosing period was 8 min, with a design loading rate of approximately 80 cm^3 min^{-1} for each orifice.

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GLOSSARY

Abbreviations

A, Ap, B, BA, BC, Bt, B/C, C, E	Soil horizon designations
ATU	Aerobic Treatment Unit
BOD_5	Five-day biochemical oxygen demand
COD	Chemical oxygen demand
CV	Coefficient of variability
EC	Electrical conductivity
K_{sat}	Saturated hydraulic conductivity
LPP system	Low-pressure pipe system
n	Number of observations
N-NH4	Ammonium nitrogen
N-NO3	Nitrate nitrogen
NSF	National Sanitation Foundation
pH	Negative logarithm of the hydrogen ion activity in solution
PVC	Polyvinyl chloride
s.d.	Standard deviation
TDR	Time domain reflectometry
0C-0-0, 0C-10-0 20L-10-0, 20R-10-0	Position of tensiomerters around an emitter in the drip irrigation experiment at the Wake County site
θ	Volumetric soil water content
%	Percent

Units of Measurement

cm d^{-1}	centimeter per day
dS m^{-1}	decisimen per meter (equal to miliequivalent per cm, meq/cm)
gal ft^{-2} d^{-1}	gallon per square foot per day
gal h^{-1}	gallon per hour
g cm^{-3}	gram per cubic centimeter
in	inch
kPa	kilo pascal
L	liter
m	meter
min	min
mg	miligram
mL	mililiter
mm	milimeter
psi	pounds per square inch
wk	week

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