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FECAL COLIFORM RELEASE STUDIES AND DEVELOPMENT OF A PRELIMINARY NONPOINT SOURCE TRANSPORT MODEL FOR INDICATOR BACTERIA

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ABSTRACT

Fecal Coliform Release Studies

The effect of grazing on water quality has been documented by bacteriological studies of streams adjacent to grazed areas. Bacterial release from fecal deposits is a parameter of the pollution transport mechanism that is poorly understood. The objective of this study was to determine a fecal coliform release function for cattle fecal deposits.

Standard cowpies were rained on with a rainfall simulator, and the fecal coliform counts were determined using the most probable number (MPN) method of enumeration. The fecal deposits were rained on at ages 2 through 100 days. The effects of rainfall intensity and recurrent rainfall were tested. Naturally occurring fecal deposits were also tested to compare their results with the results from the standard cowpies.

A log-log regression was found to describe the decline in peak fecal coliform release with fecal deposit age. The 100-day-old fecal deposits produced peak counts of 4,200 fecal coliforms per 100 milliliters of water. This quantity of release is insignificant compared to the release from fresher fecal material.

Rainfall intensity had little effect on peak fecal coliform release from fecal deposits that were 2 or 10 days old. At age 20 days the effect of rainfall intensity was significant; the highest intensity gave the lowest peak counts, and the lowest intensity gave the highest peak counts. The effect of rainfall intensity appears to be related to the dryness of the fecal deposits.

Peak fecal coliform counts were significantly lowered by raining on the fecal deposits more than once. This decline was thought to be produced by the loss of bacteria from the fecal deposits during the previous wettings.

Standard cowpies produced a peak release regression that was not significantly different from the regression for the natural fecal deposits. Apparently, grossly manipulating the fecal deposits did not significantly change the release patterns.

Modeling Studies

Grazing is a primary land use in much of the western United States, but little is known about grazing impacts on water quality. The most sensitive water quality indicators of grazing

are the fecal indicator bacteria. The objective of this study was to develop a general transport model describing the movement of fecal indicator bacteria from upland sources to channel systems.

Model development was done using simulated rainfall and a runoff surface 30.48 m by 1.83 m.

Initially the runoff surface was smooth concrete and was used to examine the effects of distance from the outlet on coliform counts by locating fecal material at various distances for five replications. Afterwards, the surface was covered by clay soil. Total and fecal coliforms were determined by the multiple-tube method.

Overland flow was described by the kinematic wave equations. Bacterial transport was modeled with a random ordinary differential equation. Initial conditions and assumptions allowed solution for the probability density function (pdf), means, and variances.

The pdf at the slope outlet was found to be normal for the assumed conditions. Solutions for the means and variances were different because initial conditions differed for the relationship between equilibrium and travel time. Three parameters fitted, a mean retention and two variance terms. The retention parameter appeared to be constant for all cases. The variance terms were obtained only for the rising hydrograph.

For the concrete surface, comparison of predicted and observed means and variances indicated poor fits during initial stages of simulation. Observed values attained steady state rapidly.

There was no replication on the soil surface, and an initial run found high background counts which were considered to be constant and were incorporated into the mean equation. A numerical solution to the mean equation was required because of the unsteady rainfall excess. The background counts and clay content of the soil prevented detection of impacts from a single source.

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INTRODUCTION

The need to determine land use impacts on nonpoint source water quality has led to modeling for inexpensive and rapid assessment. In the western United States, grazing by domestic livestock is a major land use, and there is a need to determine if and how grazing affects the quality of rangeland runoff.

One sensitive indicator of water pollution from grazed areas is the fecal indicator bacteria. Grazing animals have been found to cause an increase in concentration that is readily distinguishable from background counts. For inorganic and many other pollutants, the background variation is so great that the effects of grazing, if any, cannot be ascertained. Also, the potential health hazards of fecal pollution to humans and animals make detection of this problem particularly important.

Watershed modeling is generally divided into two phases: upland and channel. On upland zones, rainfall, soil, and vegetation characteristics are important to hydrologic response. Channel analysis is dominated by concentrated flow relationships. Livestock spend time in channels, especially on warm days, but the majority of their time is spent on the upland areas. Therefore, one hypothesis is that the greatest potential impact of cattle is from the upland areas. However, little is known about how and in what quantity organisms move from upland sources to channel systems.

Models of natural systems have twofold application. First, they provide a greater insight into the nature of key processes and the research needed to better define these processes. Secondly, the impacts of different practices can be assessed by perturbing the model parameters and estimating the change.

The objectives of this study are to define patterns of fecal coliform release from fecal material of cattle and also to develop a general transport model to describe the movement of fecal indicator bacteria from the source material to channel systems. This model, coupled with a loading function based on grazing management, would enable various grazing situations to be assessed in terms of their fecal pollution potential. If upland areas are shown to be significant contributors to fecal pollution on western rangelands, potential control practices could be evaluated, ineffective ideas eliminated, and sound approaches formulated to be more effective. If upland areas are not significant contributors, the management implication is to keep livestock away from channels.

LITERATURE REVIEW

Unconfined Grazing and Bacterial Water Quality

Several studies have been made of the effects of cattle grazing on water quality, and the most consistent finding is that bacterial pollution is the most sensitive indicator of pollution due to grazing. Other physical and chemical properties may be affected, but observed amounts cannot be separated from background variation.

In specific studies, Fair and Morrison (1967) isolated members of the Salmonella genus, potentially a pathogenic organism, from the headwaters of the Cache la Poudre River in Colorado as high as 2550.3 m (8400 ft) in elevation. Doty and Hookano (1974) reported on water quality in three high elevation watersheds in Utah which had not been grazed, logged, or burned for 45 years. Counts of fecal coliform ranged from 0-183 per 100 ml. Studies of a closed (no grazing) watershed by Walter and Bottman (1967) revealed higher counts of indicator bacteria than on an adjacent "open" watershed. Stuart et al. (1971) attributed the high counts on the closed watershed to wild animal populations that had increased to the point of creating a game preserve. McSwain and Swank (1977) examined the bacterial populations of first and second order drainages in North Carolina, and concluded that the bacteria were able to survive and even multiply in the bottom sediments of the streams.

The most obvious source of background variation in bacterial counts is the wildlife population which inhabits these watersheds. Any comprehensive modeling effort should take wildlife into account. Nevertheless, even if this is not done carefully, studies have shown that once domestic animals, cattle or sheep, are introduced, detection of grazing impacts is possible.

These studies, however, have generally focused on the stream channel. Hydrologists have long viewed the channel as an integrator of watershed processes accumulating the product consumed by the downstream water user. A natural division in the watershed modeling approach occurs at the channel. Runoff from the upland areas is modeled as shallow sheet flow, whereas in the channel hydraulic routing techniques can be used. To determine if land use practices have an impact on stream water quality, upland runoff must be modeled so that upland area contributions can be analyzed.

The following discussion examines the literature to identify problems in using channel data to assess the impacts of grazing management on bacterial pollution and to collect information on the extent of the bacterial pollution that can result from grazing domestic livestock. Also, the techniques used for enumeration of bacteria vary among standard plate counts (SPC), multiple tube or most probably number (MPN), and membrane filter (MF). Since these techniques do not measure the same populations, caution must be used when comparing studies.

Kunkle and Meiman (1967) investigated various physical, chemical, and bacterial parameters for different land uses in the upper Cache la Poudre River Basin in Colorado. Land uses included recreation and grazing, and comparisons were made with relatively unimpacted portions of the basin. Bacterial counts were made by the MF technique for two seasons. In the first season, only the total coliform group was enumerated, but the second year all three indicator groups, total coliform, fecal coliform, and fecal streptococci, were counted. The three indicator groups seem to follow a similar annual pattern of 1) flushing with high springflows; 2) post peak lulls; and 3) flushing by irrigation in July or August peaks. Grazing in an irrigated meadow resulted in increases in all indicator groups with the fecal coliform group being the most sensitive. The maximum count was 500 fecal coliform/100 ml. Of course, flushing by irrigation return flow does not represent a natural rangeland situation, and its effect on bacterial counts was not analyzed in the study. The authors used the fecal coliform to fecal streptococci (FC/FS) ratio to indicate that the source of pollution was from animals, but still questioned its applicability to the cold mountain streams. The authors concluded that bacteria counts were positively correlated with streamflow and turbidity. The correlation with turbidity was explained as a possible by-product of the turbidity-flow correlation or an indication that the bacteria were attached to the sediment.

The influence of various land-use practices on bacterial water quality in a humid region was investigated by Kunkle (1970). The land uses included forest, pasture, barns, village, and composite. The quality of water flowing from the forested area was considered the control. Crucial to examining the results of this study is the consideration of the hydrologic characteristics of the watershed. According to his analysis, variable source areas defined by saturated zones near the channels generated the runoff hydrographs. Kunkle concluded that infiltration capacities were high, therefore traditional overland flow was not observed. Overland flow distances were short, even under a snowmelt situation.

Fecal coliform counts were closely related to stream discharge from the pasture area, but plots of counts versus flow exhibited a hysteresis loop much like plots of suspended sediment versus flow on the same watershed. Given the hydrologic characteristics of the watershed, Kunkle concluded that the major sources of bacteria were the channel banks and bottom. By wading a reach of stream and agitating the bottom, an order of magnitude increase in fecal coliforms was seen 30 m downstream. The movement of a floodwave through the channels could disturb the banks and beds releasing bacteria. Kunkle concluded that this was the primary source of bacteria observed in the study. If cattle were kept away from the channel, there would be very little health hazard.

Two points need to be made about this study. First, the hydrology of the area is dominated by subsurface flow which is not conducive to bacterial transport. On western rangeland watershed, classical overland flow is more common (Gifford and Hawkins 1978). Second, the channel stores bacteria, even fecal coliforms, which can be released into the flow later. This prolongs the influence of any impact from land management, and channel bed storage must be taken into account because of potential pathogen survival.

Darling (1973), Darling and Coltharp (1973), and Coltharp and Darling (1973) reported on cattle and sheep grazing on two watersheds in northern Utah. Counts of the three indicator organisms, total coliform, fecal coliform, and fecal streptococci, followed patterns similar to those reported by Kunkle and Meiman (1967). Specifically, peaks were associated with spring flushing and with grazing activity in the summer. Once the cattle were removed from the watershed, counts declined rapidly. In this study, the cattle and sheep had access to the channels of both watersheds. Throughout the study, a stream was monitored which drained another watershed that was not being grazed, and the counts from this stream remained low and variable, but with no grazing associated peaks as on the watersheds.

The FC/FS ratio has been suggested as a means for identifying the source of fecal pollution (Geldreich 1966). A ratio of less than 0.70 indicates an animal source of pollution, and a ratio of greater than 4.0 indicates a human source. Skinner et al. (1974) studied a multiple-use watershed in Wyoming where grazing and recreational impacts were greatest during the same season of the year. MPN techniques were used for total coliform, and MF techniques were used for fecal coliform and fecal streptococci. Fecal coliform counts rarely exceeded 200/100 ml during the seasonal peak. The FC/FS ratios generally indicated the pollution to be of animal origin, but the authors indicated that these numbers should be viewed with caution because of low counts of fecal streptococci (Van Donsel and Geldreich 1971); the time required for the pollution to reach the sampling site since the ratio is valid for only the first 24 hr (Geldreich 1972); and the lack of knowledge about coliform and streptococci survival in cold mountain streams.

Concentrating animals in low elevation valleys for the winter is common in the western United States. Milne (1976) examined the chemical and bacteriological water quality impacts from this practice. He found no detectable differences in chemical quality, but bacterial counts mounted with livestock activity. Winter confinement of many animals in small areas would lead to conditions more related to feedlots than to rangeland. One advantage for management is the possibility of using point source control techniques for the operations.

The above studies examined the stream channel. Buckhouse and Gifford (1976) used rainfall simulation to study bacterial movement by overland flow and the effects of grazing on fecal pollution. Their results indicated no significant differences in fecal indicator bacteria between an ungrazed site and a site grazed at 2.0 ha/AUM. The low counts were attributed to the small percentage of the area (0.2 percent) covered by fecal material.

Also, Buckhouse and Gifford (1976) conducted a source travel distance study. By centering an infiltrometer plot over a fecal deposit and locating two plots at 0.5 m and 1.0 m distance from the centered plot, the authors hoped to determine movement associated with high intensity events. They found that at 1.0 m the fecal coliform counts averaged 23/100 ml. The conclusion reached was that the fecal coliforms were not transported further than 1.0 m on these sandy loam soils.

These results seem to confirm those of Kunkle (1970) that only areas immediately adjacent to the channel are the source of fecal pollution. Kunkle based his conclusions on the hydrology of the region in which he was working. Buckhouse and Gifford (1976) based their conclusion on the Rocky Mountain infiltrometer. In a recent discussion of sediment yield prediction, Foster et al. (1981) questioned the utility of using a small plot device such as the Rocky Mountain infiltrometer in erosion studies. Their major complaint is the lack of sufficient flow length to allow overland flow to develop and rilling to occur. Coliform transport has been linked to the erosion process.

Stephenson and Street (1978) investigated both limited grazing and intensive grazing systems at the Reynolds Creek Watershed in southwest Idaho. In the open range situation, fecal coliform counts increased after the cattle were moved onto the range and remained elevated for 3 months after the cattle were removed. There was very little correlation of physical or chemical water parameters with fecal coliforms. Rainfall-runoff events increased counts as did irrigation return flow, even when cattle were not present.

Bureau of Land Management deferred grazing system was also studied by Stephenson and Street (1978). Again, a similar pattern was observed with high counts as the cattle were moved onto the site, and when the cattle were removed, counts diminished rapidly. Still, a high intensity thunderstorm three weeks after the cattle were removed increased fecal coliform counts on the order of 200-2000 percent.

Stephenson and Street (1978) cited several factors which influence coliform counts. The major factor is the presence or absence of livestock on the watershed. Other factors include the type of event, whether rainfall or snowmelt, soils, vegetative cover, livestock density, and management practices. The latter factors are also considered important in the runoff-erosion process.

Johnson et al. (1978) studied the effects of floodplain grazing in the Colorado Front Range. An 85 ha pasture bisected by a perennial stream was grazed by 150 cows and calves from early April to mid-June in 1977. Six samples were taken prior to and after the removal of the cattle. The first sample was not taken until early June. Significantly greater values were obtained in the counts of fecal coliforms and fecal streptococci over an upstream control reach. Within 9 days following removal of the livestock, bacterial counts were not significantly different from those of the control. No runoff events were analyzed in this study which further collaborates the results of the other studies.

Doran and Linn (1979) compared a 40-ha grazed pasture to an adjacent ungrazed site. Runoff from both snowmelt and rainfall events was collected and analyzed. Fecal coliform counts in the snowmelt runoff were low since livestock were not on the pasture. High counts of fecal streptococci from the ungrazed site in snowmelt runoff was attributed to wildlife activity in a more protected area.

Analysis of the rainfall runoff data revealed fecal coliform counts were 5-10 times higher from the grazed pasture than from the ungrazed site. The fecal streptococci counts remained higher in the ungrazed area, and again, this was attributed to wildlife activity. No relationships were observed between fecal coliform and fecal streptococci and total rainfall or runoff. There was an observed relationship between air temperature, stocking density, and the counts of fecal coliform and fecal streptococci, but these factors did not account for all the variation in bacterial counts.

Doran and Linn (1979) utilized the fecal coliform to fecal streptococci ratio to differentiate between wild and domestic animals. The ratio is approximately ten times lower for wild than for domestic animals. Runoff from the grazed pasture exhibited a higher ratio than that from the ungrazed site, particularly when the cattle were present. The authors noted that caution should be used in interpreting the ratio for waste over 24 hours old, since increases in the ratio which have been observed in older waste were due to differential dieoff of fecal coliforms and fecal streptococci.

The previous discussion indicates that grazing livestock on open pasture increases the bacterial contamination of runoff water. Generally, increases are observed immediately after the animals are introduced, and counts diminish rapidly following their removal. If hydrologic conditions remain at a steady state, low counts will prevail, but disruption of the system by a runoff event, rainfall or snowmelt, will release bacteria.

Only two studies have not considered live stream channels, Buckhouse and Gifford (1976) and Doran and Linn (1979). Buckhouse and Gifford (1976) utilized a small plot rainfall simulator with high intensity events and concluded that fecal coliform did not travel over 1.0 m. Doran and Linn (1979) indicated that cattle were kept at least 10.0 m away from the outlet of the pasture, and they were able to determine an increase in fecal coliforms in runoff from natural rainfall events. The main limitations to Buckhouse and Gifford's results as noted above, is that their method did not incorporate rill erosion.

Overland flow, though a small portion of water yield from most watersheds, is the predominant mode of bacterial transport from upland source areas. At present, no information on source-distance and transport relationships is available, and these must be established for contributions from upland areas to be modeled. In addition fecal material in and adjacent to the channel needs to be considered.

Coliform Modeling

Previous efforts at modeling coliform bacteria have centered on channel and estuary systems. Canale et al. (1973) tried time series, multiple regression, and mass balance models for modeling total coliforms for a bay on Lake Michigan for several applications. Though the time series and regression models had high R^2 (0.88 and 0.89, respectively), they were considered adequate only for short-range forecasting, and the mass balance model was preferred for long-range forecasting. For modeling coliform survival, they used first order rate kinetics and discounted any coliform growth, since the bay was low in bacterial nutrients.

Canale et al. (1973) divided the Lake Michigan estuary into a zone with complete mixing and a zone with one-dimensional flow. For the zone with complete mixing, the continuity of coliforms was

$$\frac{dc}{dt} = \frac{1}{-T} + K(T)\ c + \frac{W(t)}{v} \quad . \quad . \qquad (1)$$

where

c = coliform counts
T = average residence time of a fluid element in the zone
$K(T)$ = first order reaction rate, which is temperature dependent
v = volume of the zone
$W(t)$ = coliform loading in time
t = time

For the zone with flow, the continuity equation is

$$\frac{\partial c}{\partial t} = E_x \frac{\partial^2 c}{\partial_x 2} - U_x \frac{\partial c}{\partial x} - K(T)c \quad . \qquad (2)$$

where

E_x = dispersion coefficient
U_x = velocity of flow

The dispersion term is required for the estuary, but in stream situations, it is generally ignored.

Mahlock (1974) compared applications of deterministic and statistical models to both total and fecal coliform data from the Leaf River in Mississippi. He was able to calibrate a deterministic model that described total coliforms but did not incorporate the environmental factors needed to describe fecal coliforms.

White and Dracup (1977) modeled fecal coliform in a high mountain watershed by applying Equation 2 without the dispersion term ($E_x = 0$) to steady state conditions. First-order rate reactions were used to describe the decay of coliforms, and a sensitivity analysis of the reaction coefficient indicated that it had no effect on the results because of the short travel time of the stream. Given this, the authors concluded that fecal coliform concentrations could be expected to move through this system undiminished from headwater values. Evidently, this conclusion was for the steady-state system because data from rainfall events were not considered.

Kay and McDonald (1980) employed a logarithmic dieoff of bacteria based on distance rather than time. Since they were modeling relatively slow water movement through reservoirs, the dieoff would be much greater than in streams, particularly during summer stratification. But the essential idea of their method, using distance, seems to be superior because one does not have to make assumptions about retention time. The approach could also be applied to the stream modeling.

Coliform modeling is not as advanced as is modeling movements of other water quality parameters. The studies reviewed above all centered on first order kinetics to describe the loss of coliforms in lake or stream situations. Except for Canale et al. (1973), the modelers only considered steady-state conditions and point source inputs. No attempt has been made to model coliform inputs into stream systems from watersheds. A recent review by the Forest Service (1977) indicated that there were no models to describe bacterial movement through wildland watersheds.

PART I

FECAL COLIFORM RELEASE STUDIES

OBJECTIVES

The objectives of this study were to determine for rainstorm events the peak rates of fecal coliform release from cattle feces and how these rates vary with fecal-deposit age, recurrent rainfall events, different types of fecal deposits, and varying rainfall intensities.

MATERIALS AND METHODS

These studies were conducted during the summers of 1980 and 1981 at Utah State University, Green Canyon Research Area. This area is located at the mouth of Green Canyon in North Logan, at an elevation of 1400 meters.

Rainfall Simulation

The rainfall was produced by a rainfall simulator designed by Meeuwig (1971) and modified by Malekuti (1975). This rainfall simulator uses 518 stainless-steel needles to produce drops (Figure 1). The needles protrude from the bottom of a 0.58 by 0.58 meter chamber. The water chamber allows for 12.5 millimeters of water to pond above the needles. The rainfall simulator has a flow meter and a valve to regulate the flow of water from a 18.9-liter reservoir to the water chamber; regulation of the flow regulates the rainfall intensity. An electric motor rotates the water chamber to give a more evenly distributed rainfall pattern.

The simulator was elevated to a height of 3.7 meters, and the raindrop fall area was protected by a canvas windscreen. A collection board was placed under the rainfall simulator. The collection board rested on a stand that gave the board a 10 degree slope; this caused all the runoff to flow towards the collection trough.

The collection board was a 0.61-meter by 0.61-meter by 25.4-millimeter piece of plywood. A collection trough was attached to one edge of the board, and the trough sloped to the center where a drain hole was located (Figure 2). A 25.4-millimeter-high metal rim was attached to the top of the board to give it a 0.168-square-meter, splash, runoff collection area.

The simulated rainfall drop size was determined by randomly collecting 100 drops in a 10-milliliter, graduated cylinder. This was done 10 times, and the average drop was determined to be 0.0139 milliliters; this volume was calculated to have a spherical diameter of about 3.0 millimeters. The rainfall simulator was 3.7 meters above the collection board, so the simulated rainfall reached 80 percent of terminal velocity (Laws 1941).

The rainfall simulator was calibrated using a 203 millimeter rain gage. The rain gage was placed on that portion of the collection board where the fecal deposits were normally placed. The rainfall simulator was calibrated daily at the rainfall intensities to be used that day.

Fecal Deposits and Sampling

The fecal deposits used in this study were either naturally-occurring fecal deposits or "standard cowpies" (described later). All of the cattle feces were obtained from Hereford heifers at the Poison Plant Research Laboratory, and then transported to the Green Canyon Research Area for aging. Only feces from control heifers which had been fed a diet of alfalfa hay with mineral supplements were used.

The average weight of a fresh, naturally-occurring fecal deposit was determined by weighing 100 such deposits after removing such extraneous debris as rocks and straw. The deposits were placed on a weighing platform, with

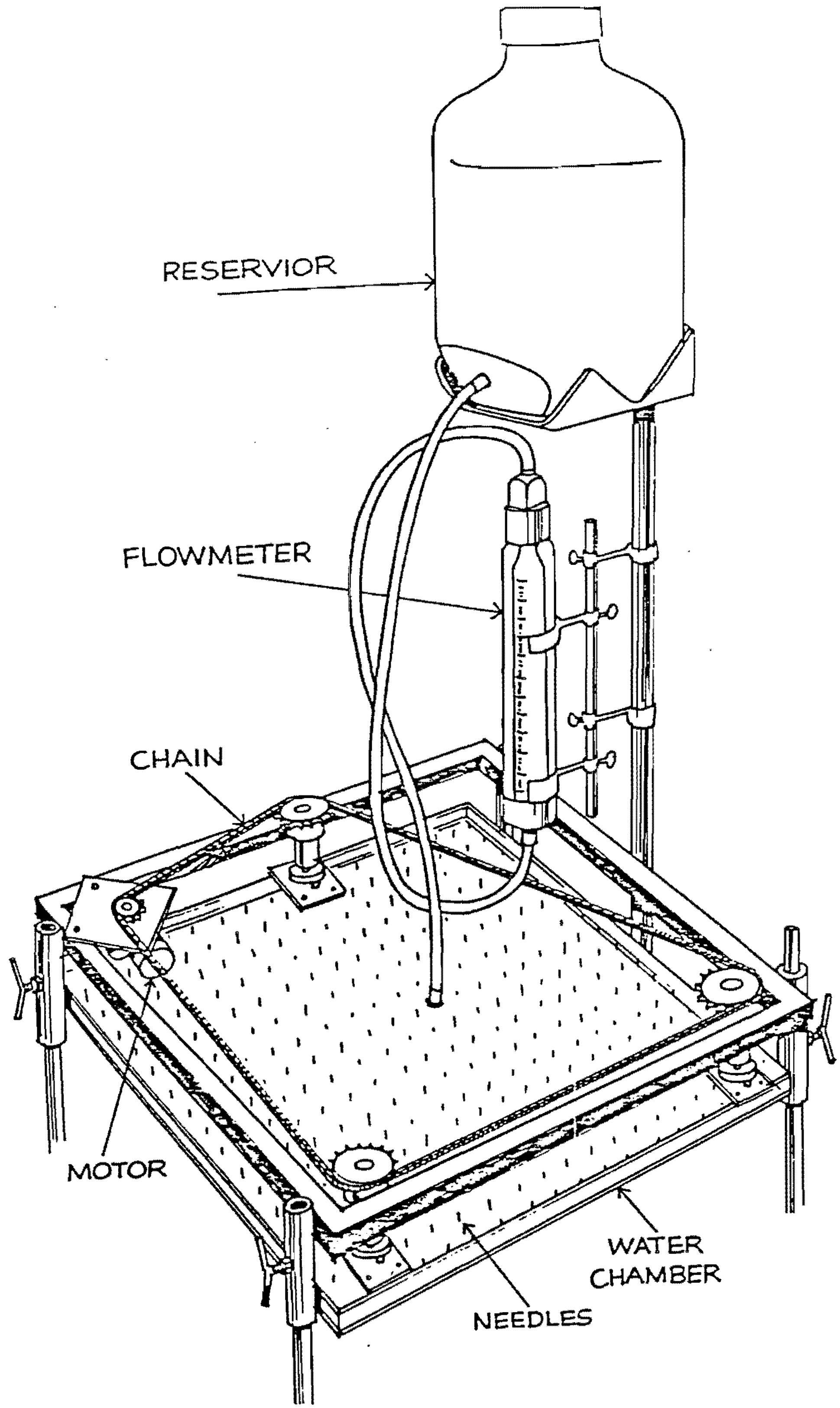

Figure 1. Rainfall simulator.

the use of a trowel, and weighed with a spring scale. The mean weight was determined to be 1.24 kilograms, so any naturally-occurring fecal deposit that had a fresh weight within 0.113 kilograms (one standard deviation) of the mean weight was considered eligible to be used as a natural dung pile.

The standard cowpies were made by collecting fresh fecal material, mixing it in a cement mixer for 15 minutes, weighing out 0.907 kilograms of the mixed feces, and forming the cowpie in a 0.203-meter cake pan.

The fecal deposits were transported to the Green Canyon Research Area. There, they were placed on a bare, mineral soil that had been covered with a thin layer of sand. The fecal de-

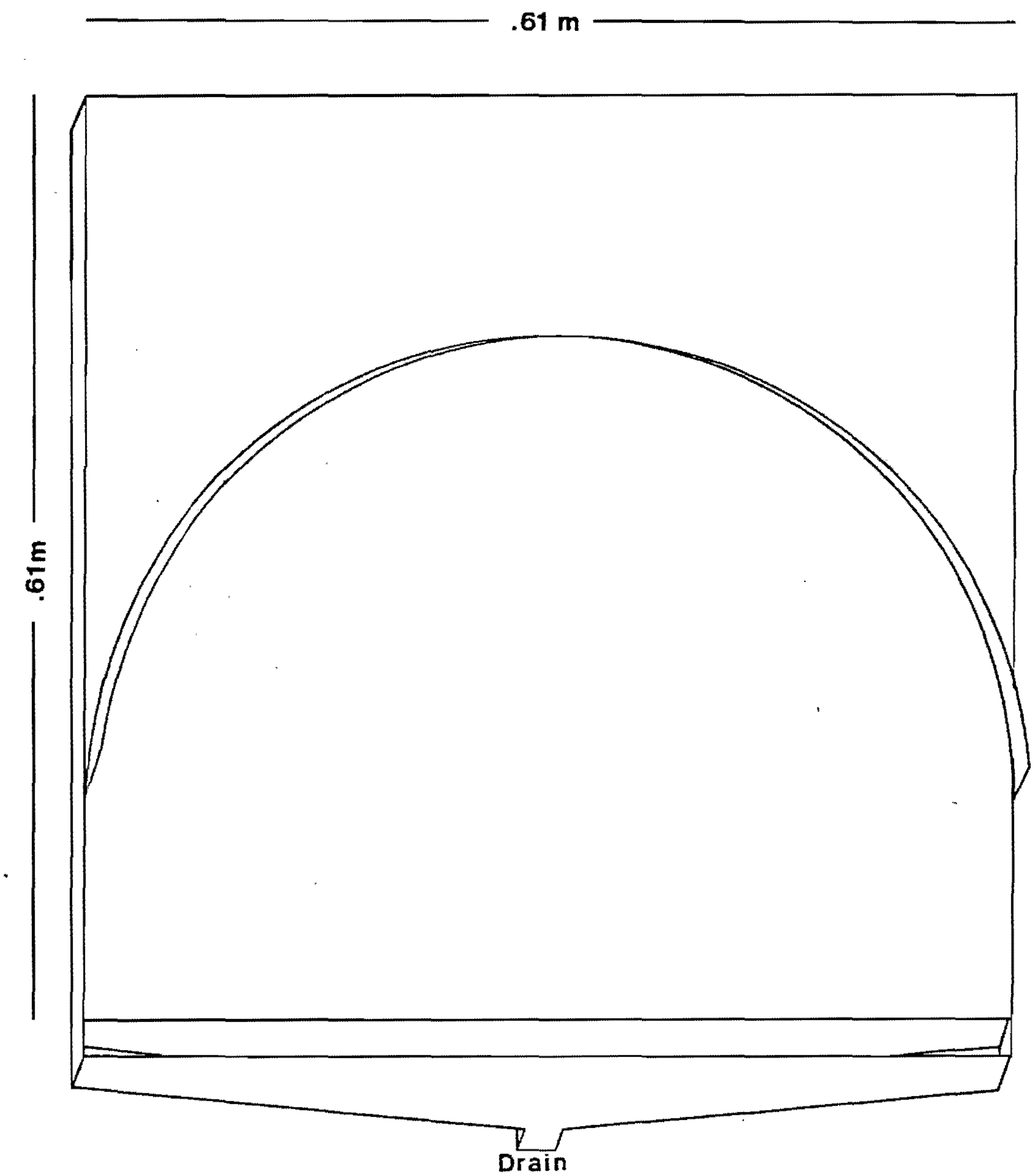

Figure 2. Platform used for collecting runoff from fecal deposits during rain simulations.

posits were covered with plastic tarpaulin when natural rain occurred.

A total of 117 "pies" were initially tested, when they were fresh and at 2, 3, 4, 5, 10, 15, 20, 25, and 30 days. The sampling series began on the first day with three fresh fecal deposits being tested. On the succeeding days, one run on a fresh deposit was conducted followed by three runs on a specific age deposit. The entire series was repeated three times. All simulated rainfalls on the fecal deposits were at a rate of 6.1 ± 0.3 cm hr^{-1} for 15 minutes.

Runoff samples were collected over a 30 second interval at 5, 10, and 15 minutes in whirl-pak bags. A 30 second sample interval was necessary to obtain a sufficient volume of sample water. Collection was initiated 15 seconds prior to the 5, 10, or 15 minute mark and completed 15 seconds after. Once collected, the bags were immediately placed in a styrofoam cooler for transport to the lab within a few hours.

Upon completion of each test run, the fecal deposit was either returned to its sand pad for additional aging or discarded. The platform was cleaned and disinfected by scrubbing with chlorine bleach and rinsing with a 10 percent solution of sodium thiosulfate. In order to check this cleaning procedure, a blank run was made on three occasions. This involved raining on the empty, but just cleaned, platform and taking samples at 5, 10, and 15 minutes as usual.

To meet study objectives and to expand on the initial study, a second study phase consisted of six treatments. There was one 100 day treatment; there was one dung-pile treatment; there was one recurrent-rainfall treatment; and there were three rainfall-intensity treatments. In addition to the six treatments, background counts were also obtained.

The 100 day treatment was a longevity study. It served as the control for the recurrent-rainfall and the dung-pile treatments, and it was also used as the mid-intensity treatment. Each of 72 cowpies was tested with one simulated rainfall and then discarded. These cowpies were rained on at age 2, 10, 20, 30, 40, 50, 70, or 100 days.

The dung-pile treatment was designed to determine if there were significant differences between natural fecal deposits and standard cowpies. The dung-pile treatment used 54 natural dung piles; each dung pile underwent one simulated rainfall and was then discarded. The dung piles were rained on at age 2, 10, 20, 30, 40, or 50 days.

The recurrent-rainfall treatment was used to determine the effect of raining on fecal deposits more than once. Nine cowpies were used for the length of the treatment. The nine cowpies underwent a simulated rainfall at ages 3, 10, 20, 30, 40, and 50 days.

The intensity treatments were designed to determine if rainfall intensity was a significant factor in fecal coliform release. Three rainfall intensities were used; these were 23, 51, and 69 millimeters per hour; all other treatments used a rainfall intensity of 51 millimeters per hour. Each of the intensity treatments used 27 standard cowpies; each cowpie underwent one simulated rainfall and was then discarded. These cowpies were rained on at age 2, 10, or 20 days.

Each treatment age required nine fecal deposits. Three of the fecal deposits were rained on for 25 minutes each, and the runoff was sampled at 5 minute intervals. The time of peak FC release was determined. The six remaining fecal deposits were then rained on, but they were sampled only at the time of predetermined peak release.

Seven blank runs were taken to determine background counts. A clean collection board was placed under the rainfall simulator, and a sample was taken after 5 minutes of rainfall and tested just like the other samples.

After each sample run, the collection board was rinsed with water, cleaned with chlorine bleach, rinsed with water again, rinsed with a solution of 0.02 percent sodium thiosulfate, and set in the sun to dry. This process was for disinfection between runs.

The water used in the rainfall simulator was from North Logan's drinking water supply. Four milliliters of a 10 percent solution of sodium thiosulfate were added to each gallon of water to remove any possible chlorine.

Bacterial Analysis

Upon completion of the day's test runs, the samples were brought back to the lab for immediate bacteriological analysis. The MPN method (American Public Health Association 1975) was used instead of the membrane filter method for two reasons. First, the close proximity of the sample deposit to the runoff collection point resulted in substantial organic debris being present in the sample water. The amount of debris was sufficient to clog the membrane filters. The MPN method proved to be the only practical method for this study. In addition, and for essentially the same reason, other phases of the overall project also used the MPN method. In order to have compatible results, the MPN method was selected for all phases of the project, including this study.

The MPN method for counting fecal coliforms combines a presumptive test and a confirmed test. The presumptive test uses five tubes at each of five dilutions per sample, for a total of 25 tubes of lauryl sulfate tryptose (LST) lactose broth, inoculated with samples of the runoff water and incubated at 35°C. The series of dilutions used depended upon the unrained-on age of the fecal deposit being tested. The youngest ones required dilutions from 10^{-3} to 10^{-7}. The older deposits and some recurrent rain tests required dilutions ranging from 10° to 10^{-4}. Lauryl sulfate is a surface active detergent which inhibits growth of gram-positive organisms while encouraging growth of coliforms. Coliforms use up any oxygen present in the broth and then ferment the lactose producing acid and gas under anaerobic conditions. Gas formation in 24 or 48 hours is a positive test (Kelley and Post 1978).

LST tubes showing a positive test for gas at 24 or 48 hours were subcultured into tubes of EC broth and incubated at 44.5°C in a water bath to determine the presence of fecal coliforms. EC medium contains bile and lactose. The bile inhibits gram-positive bacteria while the high temperature selects only those organisms able to grow at this temperature. Gas in 24 hours is a positive test for fecal coliforms (Kelley and Post 1978).

The patterns of positive EC results obtained were used to determine the most probable number (MPN) by consulting the five tube, three dilution table in Standard Methods (American Public Health Association 1975). The MPN's are statistically derived and indicate the most probably number of bacteria producing the observed pattern of results. All final results were recorded as fecal coliform counts per 100 ml of runoff water.

RESULTS

Effect of Unrained-on Age on Coliform Release (Initial Studies)

The data obtained were highly variable. After 5 minutes of rainfall, the counts from some of the older deposits were as low as 20 per 100 ml. On the other hand, some samples taken from 2 day old deposits after 10 minutes of rainfall gave counts as high as 4.9×10^7 per 100 ml. This high variability was also found within individual sample groups. Values ranged from as low as 20 to 10^6 per 100 ml within one age and rainfall duration group.

The data were analyzed in two ways. Regression analysis was used to determine the relationship between cowpie age and the number of coliforms washed from it at each of the three measurement times after rainfall began. Analysis of variance was used to examine the data for significant differences among combinations of age and sampling time. Both analyses were based on the logarithms (base 10) of the data to minimize the effect of the variability in the measurements and because coliform MPN counts tend to be log normally distributed.

Regression was used to determine the effect of aging on the coliform count for the 5 minute, 10 minute, and 15 minute sample means (the mean $\log_{10}$) fecal coliform count per 100 ml). The regression for the 5 minute means was the only one that resulted in a linear equation (Figure 3).

Adding the terms $(\log_{10} X)^2$ and $(\log_{10} X)^3$ did not significantly, as determined by tests described in Snedecor and Cochran (1969), improve the fit. This is most likely due to the greater variability found in the samples taken at 5 minutes. Even so, the fit is quite good with an R^2 value of 0.937.

A better fit was obtained for the 10 minute means with a curvilinear expression (Figure 4). Multiple regression techniques (Snedecor and Cochran 1969) provided a cubic equation with a significantly better fit than either a quadratic or linear equation. The fit was even better than the 5 minute analysis with an R^2 of 0.969.

Unlike the negative exponential curve of the 5 minutes analysis, the 10 minute curve shows an initial increase during the first 2 days of aging. Afterwards, the mean MPN follows a decline similar to that for the 5 minute samples with a tailing off that suggests very little cowpie aging after 30 days.

The 15 minute data produced a curve very similar to that of the 10 minute data (Figure 5). Once again, the best fit was by a cubic equation. The R^2 value was 0.991.

Since the 10 and 15 minute regression curves cannot be differentiated statistically (as described below in the analysis of variance testing), it seemed appropriate to pool the data for each age from the two sampling times and repeat the regression analysis. The resulting regression curve is in Figure 6. The equation is cubic with an R^2 of 0.983.

Analysis-of-variance testing was used to examine significances:

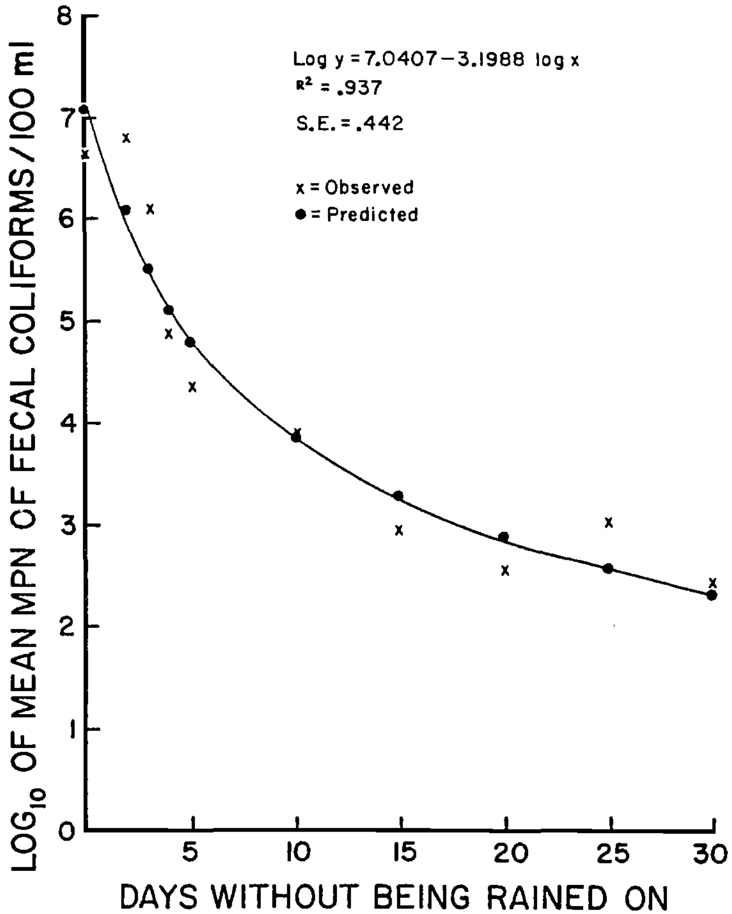

Figure 3. Regression curve for the 5 minute (following start of rainfall) sample means.

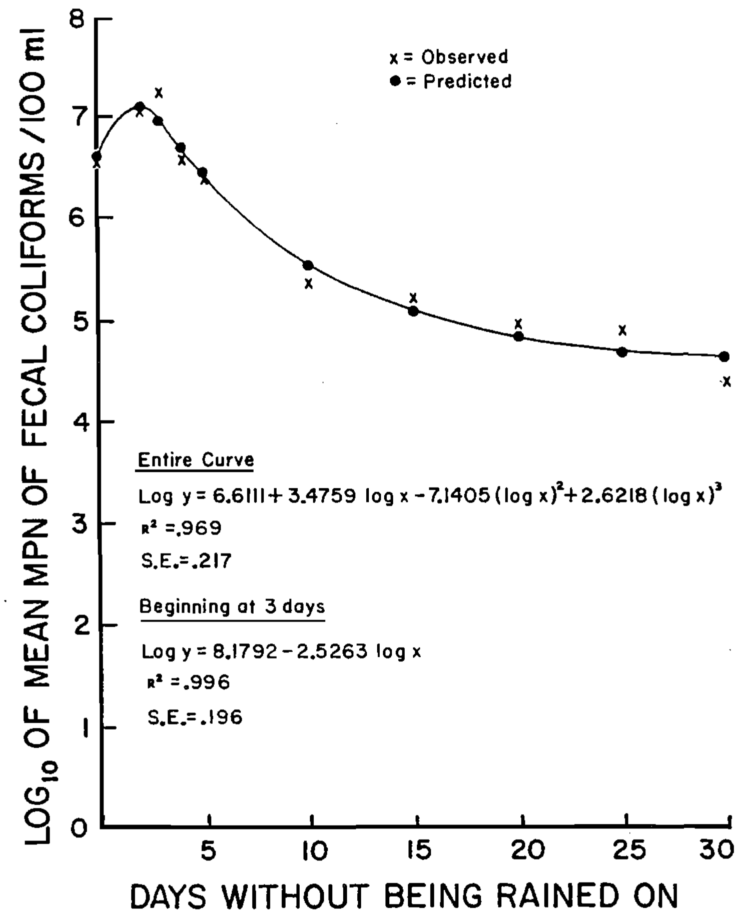

Figure 4. Regression curve for the 10 minute (following start of rainfall) sample means.

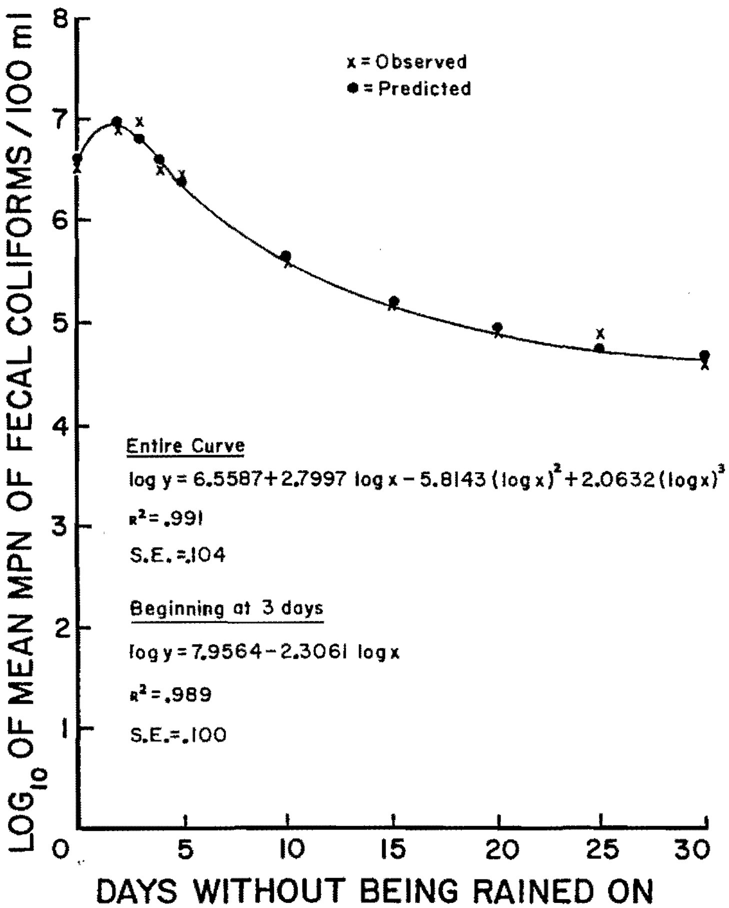

Figure 5. Regression curve for the 15 minute (following start of rainfall) sample means.

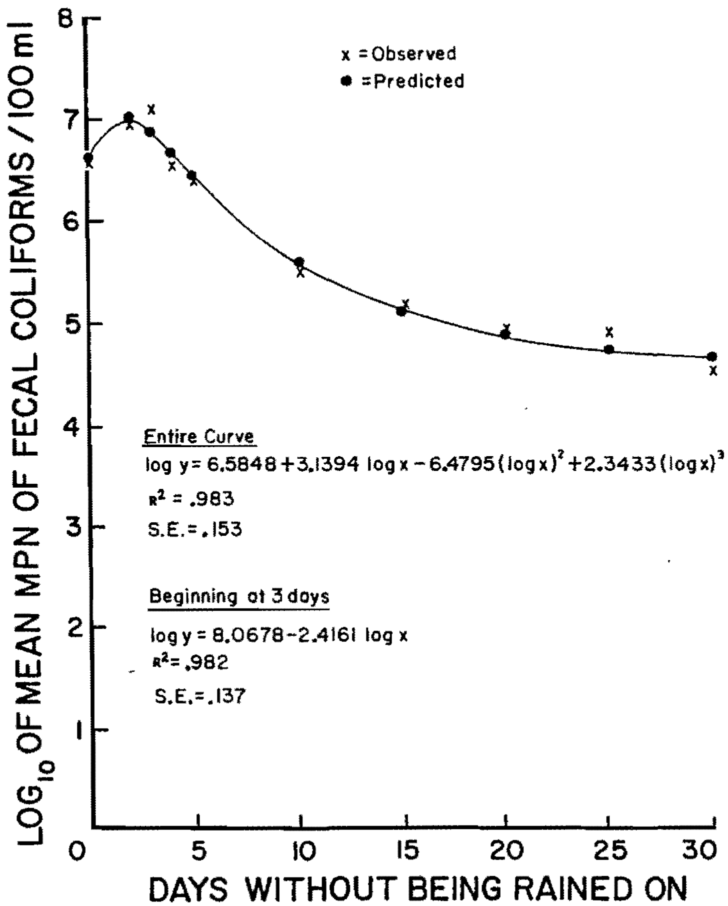

Figure 6. Regression curve for the pooled 10 and 15 minute (following start of rainfall) sample means.

1. Between unrained-on age at each sampling time.
2. Between sampling times at each unrained-on age.
3. Between replications at each unrained-on age and sampling time combination.

The method described in Ott (1977) for a one-way classification test of a hypothesis about t>2 population means was used. When the F test was determined to be significant, the sample means were examined by Fisher's Least Significant Difference test to determine which population means differed.

The analysis of variance test showed a significant (at the 0.05 level) difference in coliform count with cowpie age at all three sampling times as well as with the pooled 10 and 15 minute data.

Table 1 lists the sample means and variances. Since each of the four groups exhibited significant differences in means as the sample aged, Fisher's LSD test was carried out on each group. Table 2 lists the results.

As can be seen from Tables 1 and 2, there is more variation and also more significant differences among sample means at the 5 minute sample time. It could be that as the fecal deposits age they require additional time for thorough wetting and therefore take longer to reach an equilibrium fecal coliform release rate.

The fact that this wetting period is not needed for fresher fecal deposits may account for the lack of a hump in the 5 minute regression curve (Figure 3). At 5 minutes the fresh and 2-day-old means are not significantly different, as is the case with the 15 minute means. At 10 and 15 minutes as well as with the 10 and 15 minute pooled means, the coliform release rates for the fresh cowpies are always significantly less than for the 3-day-old deposits.

The 5 minute samples for the nine 25-day-old cowpies showed the greatest variation of any of the age and time combinations (Table 1). This was mainly due to one sampling having a very high MPN value. Why this occurred is not known. However, this example points out the large amount of variability and the high fecal coliform yield potential of the older fecal deposits after only 5 minutes of rainfall.

When the 10 and 15 minute analyses are pooled, the effect of unrained-on age on fecal coliform release can be broken into five groups. Group I consists of the 2 and 3-day-old deposits. These release the greatest number of fecal coliforms. Group II follows with the fresh, 4 and 5-day-old deposits not being significantly different from each other. From there we go to Group III, 10 and 15-day-old deposits. There is some overlap here with Group IV which is composed of 15, 20, and 25-day-old deposits. Finally, Group V consists of the 30-day-old deposits.

The difference between these groups is, in all cases but one, a drop of approximately one-half an order of magnitude. The exception is the drop of slightly more than one order of magnitude between Groups II and III. These groupings illustrate that the rate of release of fecal coliforms does not decline significantly within a day or two (except within the first 5 days) but instead takes 5 to 10 days or more. This finding has significant land management implications.

The second application of the analysis of variance test was to look for significant differences among sampling times at each unrained-on age. The fresh and 2-day-old deposits showed no significant difference, at the 0.05 level, among the 5, 10, and 15 minute sample means. The rest of the age groups all had 5 minute means significantly less than the 10 and 15

Table 1. Statistics for AOV between days and sampling times. The means are the mean $\log_{10}$ MPN.

Days	Mean	Variance	N
5 Minute Samples			
Fresh	6.636	0.168	36
2	6.697	0.335	9
3	6.076	0.353	9
4	4.861	0.266	9
5	4.321	0.099	9
10	3.871	0.816	9
15	2.924	0.669	9
20	2.554	0.281	9
25	3.027	1.280	9
30	2.402	0.588	9
10 Minute Samples			
Fresh	6.582	0.165	36
2	7.051	0.168	9
3	7.219	0.049	9
4	6.554	0.090	9
5	6.371	0.151	9
10	5.349	0.650	9
15	5.198	0.564	9
20	4.938	0.482	9
25	4.887	0.429	9
30	4.378	0.220	9
15 Minute Samples			
Fresh	6.564	0.172	36
2	6.865	0.131	9
3	6.946	0.128	9
4	6.484	0.205	9
5	6.404	0.230	9
10	5.580	0.833	9
15	5.148	0.353	9
20	4.888	0.303	9
25	4.881	0.230	9
30	4.571	0.385	9
10 and 15 Minute Pooled Samples			
Fresh	6.573	0.166	36
2	6.958	0.149	9
3	7.083	0.103	9
4	6.519	0.142	9
5	6.387	0.180	9
10	5.465	0.712	9
15	5.173	0.432	9
20	4.913	0.370	9
25	4.884	0.310	9
30	4.475	0.248	9

Table 2. Results of Fisher's LSD test for AOV between unrained-on age (days) and sampling time.

These Means Are	Significantly Less Than	These Means
5 Minute Samples		
30, 20, 15, 25, 10, 5, 4, 3		Fresh, 2
30, 20, 15, 25, 10, 5, 4		3
30, 20, 15, 25, 10		4
30, 20, 15, 25		10, 5
30		25
10 Minute Samples		
30, 25, 20, 15, 10, 5, 4, Fresh		2, 3
30, 25, 20, 15, 10		5, 4, Fresh
30		25, 20, 15, 10
15 Minute Samples		
30, 25, 20, 15, 10, 5, Fresh		3
30, 25, 20, 15, 10		5, 4, Fresh, 2
30, 25, 20		10
30		15
10 and 15 Minute Pooled Samples		
30, 25, 20, 15, 10, 5, 4, Fresh		2, 3
30, 25, 20, 15, 10		5, 4, Fresh
30, 25, 20		15, 10
30		25, 20

minute means. Table 3 lists the sample means and variances.

These results help explain the lack of a hump in the 5 minute curve. They also support the hypothesis that it takes longer than 5 minutes to reach equilibrium in coliform release rates from deposits 3 days old and older. Equivalent values for the 10 and 15 minute samples show that an equilibrium has indeed been attained by 10 minutes even for the 30 day cowpies.

Three blank runs were made during the study. These results are listed in Table 4. Ideally, all coliform counts from this process would have been zero. It is possible that some fecal coliforms were in the water used in the rainfall simulator. Another possible source could be splash from the ground surrounding the empty platform. However, the most likely source was airborne dust.

Whatever the actual source of these fecal coliforms found on the blank runs, the amount is small and insignificant when compared to the samples having millions of fecal coliforms per 100 ml. Some of the individual replications of the older age deposits at the 5 minute sampling time did, however, produce results that could have been influenced by counts of the size measured in the blank tests. However, given the mean values obtained, the cleaning procedure seems sufficient.

The fecal coliform concentrations measured in this study should not be taken as the actual concentrations of bacteria being released from the fecal

Table 3. Statistics for AOV between sampling times and days. Time is in minutes and the means are the mean $\log_{10}$ MPN.

Time	Mean	Variance	N
Fresh Deposits			
5	6.636	0.168	36
10	6.582	0.165	36
15	6.564	0.172	36
2-Day-Old Deposits			
5	6.697	0.335	9
10	7.051	0.167	9
15	6.865	0.131	9
3-Day-Old Deposits			
5	6.076	0.353	9
10	7.219	0.049	9
15	6.946	0.128	9
4-Day-Old Deposits			
5	4.861	0.226	9
10	6.562	0.085	9
15	6.484	0.209	9
5-Day-Old Deposits			
5	4.321	0.099	9
10	6.371	0.151	9
15	6.404	0.230	9
10-Day-Old Deposits			
5	3.871	0.816	9
10	5.349	0.650	9
15	5.580	0.833	9
15-Day-Old Deposits			
5	2.924	0.669	9
10	5.198	0.564	9
15	5.148	0.353	9
20-Day-Old Deposits			
5	2.554	0.281	9
10	4.938	0.482	9
15	4.888	0.303	9
25-Day-Old Deposits			
5	3.028	1.282	9
10	4.887	0.429	9
15	4.882	0.230	9
30-Day-Old Deposits			
5	2.402	0.588	9
10	4.378	0.220	9
15	4.571	0.385	9

Table 4. MPN results from blank tests.

Sample Time	MPN of Coliform per 100 ml			
	Run One	Run Two	Run Three	$\bar{x}$
5	23	4	23	17
10	79	2	33	38
15	70	2	33	35

deposits. Some dilution takes place before the sample is collected. The impervious platform is, in a sense, a small watershed from which runoff is 100 percent of the rainfall. The fecal deposit does not cover the entire area. Even with coliforms coming from splash in this zone dilution is occurring. Therefore, the actual fecal coliform concentration is higher than what was achieved. By varying the percentage of runoff area that the deposit occupies, one will get varying, but proportional, concentrations of fecal coliforms being released. In this study, the fecal deposit occupied 15.6 percent of the runoff area.

Effect of Age on Coliform Release (Second Phase)

The 100 day aging treatment showed a decline in peak FC release with fecal-deposit age (Table 5). The peak, transformed, FC counts decreased at a nearly constant rate for the first 50 days of aging (Figure 5); the coefficient of determination (R^2) for a log linear regression covering this period was 0.973. The rate of decline then leveled off, and the slope of the line from day 50 to day 100 was not significantly different from zero (Figure 7).

A log-log transformation of both the peak FC and aging time data over the entire 100 day aging period yielded a regression with a R^2 of 0.923 (Figure 8). Two of the data points that determined the regression fell outside of the 95 percent confidence interval. Only one point fell outside the confidence interval for the log-normal regressions (Figure 5).

Peak FC counts from the initial studies were compared to both regressions in this second phase. For the log linear regression over 50 days of aging, the first four points fell above and the last five points fell below the regression line (Figure 5). Only two points fell within the confidence interval of the log-normal regression. All peak FC points fell within the confidence interval of the log-log regression (Figure 8).

FC regressions were also run for each 5 minute time interval over the 100 day aging treatment. The point of slope change for the log-normal analysis varied with sampling time; the 5 minute and 10 minute samples changed slope around day 10 (Figures 9 and 10), while the 15, 20, and 25 minute samples changed slope around day 50 (Figures 11, 12, and 13). The log-normal regressions that came after the change in slope were not significantly different from a slope of zero, except at the 25 minute interval and the 25 minute interval had only two points available for determining the second regression.

The log-normal regressions were then compared to the log-log regressions. The log-normal regressions had higher R^2 values than the log-log regressions at the 15 and 25 minute intervals; during the 5, 10, and 20 minute intervals the reverse was true. The log-normal regressions had a high R^2 value of 0.991 at the 15 minute interval and a low R^2 value of 0.013 at the 5 minute interval. The log-log regressions had a high R^2 value of 0.938 at the 15 minute interval and a low R^2 value of 0.669 at the 5 minute interval.

From the initial study, 5, 10, and 15 minute data were compared to their

Table 5. Runoff from standard cowpies was collected at 5 minute intervals, and the fecal coliform counts were determined (raw data were transformed to the $\log_{10}$).

Age Days	Mean FC/100 ml	Variance	N
Peak Counts			
2	6.69	0.155	9
10	5.73	0.146	9
20	5.53	0.082	9
30	4.95	0.398	9
40	4.42	0.328	9
50	3.84	0.492	9
70	3.68	1.54	9
100	3.62	0.356	9
5 Minute Counts			
2	5.38	0.070	3
10	2.70	0.262	3
20	2.79	0.609	3
30	2.67	0.113	3
40	2.07	0.501	3
50	3.02	0.802	3
70	5.07	0.327	3
100	2.57	0.482	3
10 Minute Counts			
2	6.69	0.155	9
10	3.65	0.361	3
20	4.68	0.051	3
30	3.88	0.794	3
40	4.42	0.328	9
50	3.69	0.436	3
70	4.97	1.06	3
100	2.96	0.697	3
15 Minute Counts			
2	6.84	0.109	3
10	5.73	0.146	9
20	5.53	0.082	9
30	4.95	0.398	9
40	4.33	0.685	3
50	3.84	0.492	9
70	3.68	1.54	9
100	3.35	1.61	3
20 Minute Counts			
2	7.19	0.077	3
10	4.85	0.333	3
20	5.76	0.192	3
30	4.92	1.12	3
40	3.70	0.502	3

Table 5. Continued.

Age Days	Mean FC/100 ml	Variance	N
20 Minute Counts continued			
50	3.77	0.048	3
70	5.31	0.366	3
100	3.62	0.356	9
25 Minute Counts			
2	5.91	0.338	3
10	5.06	0.675	3
20	5.59	0.076	3
30	4.86	0.355	3
40	3.98	0.447	3
50	3.85	0.036	3
70	5.21	0.178	3
100	3.40	0.664	3

respective regressions in this study. All of the data points fit within the confidence interval of the appropriate log-log regression, except for days 2 and 3 of the 5 minute regression and day 25 of the 10 minute regression (Figures 14, 15, and 16).

The 70 day counts were not used in determining the 5, 10, 20, or 25 minute, 100 day regressions (Figures 9-18). The three, 70-day-old fecal deposits that were sampled at 5 minute intervals gave peak counts that were significantly higher than those for the other samples. These atypical points also fit poorly into the regressions (Figures 9-18); their calculated values were more than four standard deviations from their predicted values. Counts from the three atypical cowpies were used in regression determination when they could be averaged with the counts from six other cowpies, but they were not used when they had to be relied on by themselves.

Effect of Recurrent Rainfall

The peak FC counts from fecal deposits subjected to recurrent-rainfall treatment were consistently lower than their once-wet counterparts (Figure 19 and Table 6). The decline was significant on the third, fourth, and fifth wettings but not significant on the second or sixth.

The peak FC counts of the recurrent-rainfall treatment were compared to the peak, log-normal regression of the once-wet treatment (Figure 20). The count with the rewet treatment declined at a greater rate than did the once-wet treatment for the first 20 days; however, the rate of decline for the rewet treatment fluctuated for the next 30 days.

The rewet treatment was analyzed with the assumption that the effect of age was the same regardless of the number of wettings; this separated the effect of age from the effect of the rewettings. The rewet treatment was also analyzed without the age-effect assumption. Log-normal and log-log regressions were run for the rewet treatment and compared to the log-normal and the log-log regressions for the once-wet treatment (Figures 21 and 22). The log-normal, rewet regression had an R^2 of 0.769, and its slope was not significantly different from the slope of the once-wet regression (Figure 21).

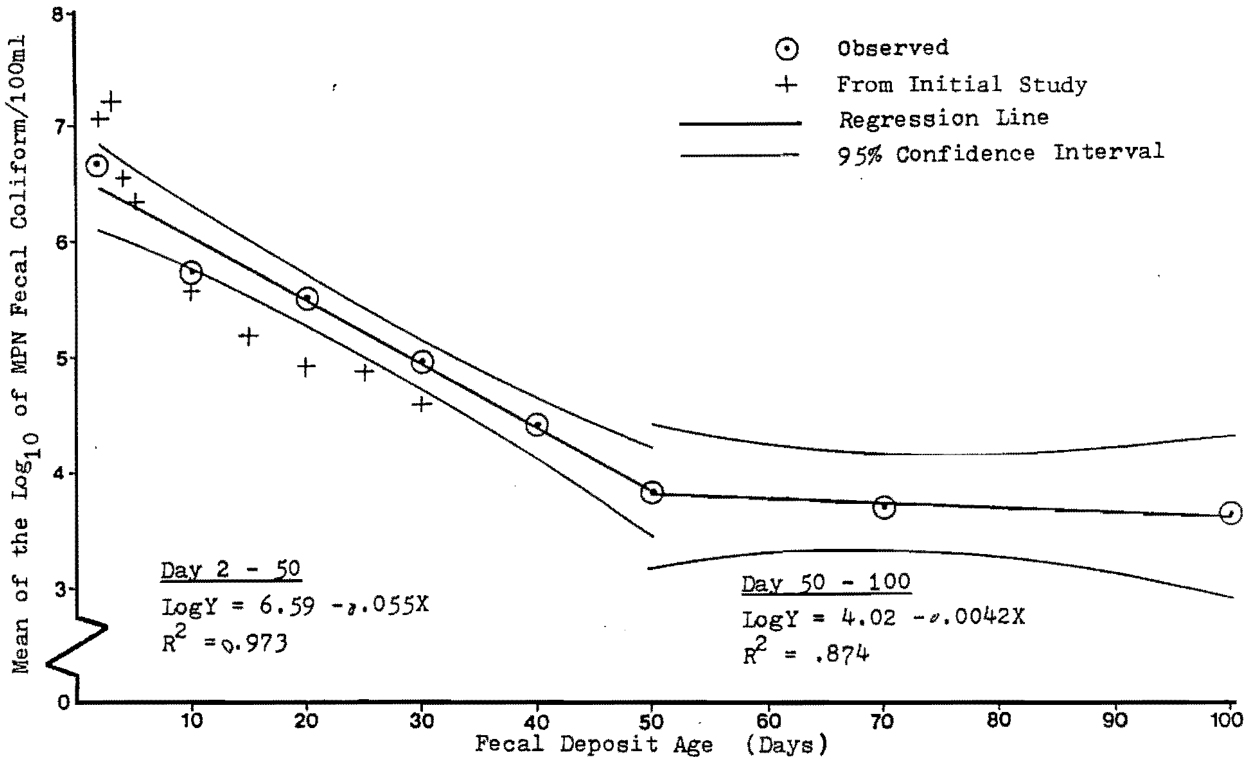

Figure 7. The two, log-normal regressions were calculated from the peak fecal coliform release from standard cowpies. Peak data from the initial study are also shown.

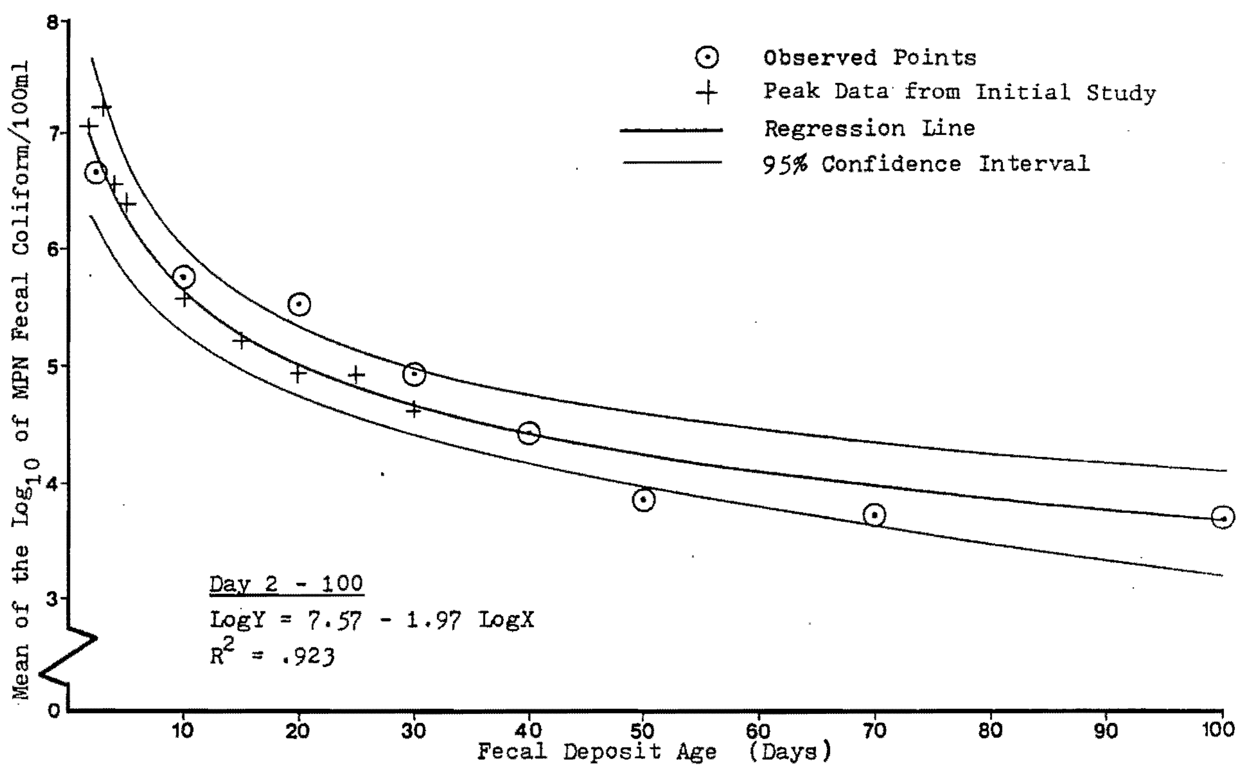

Figure 8. The 100 day, log-log regression was calculated from the peak fecal coliform release from standard cowpies. Peak data from the initial study are also shown.

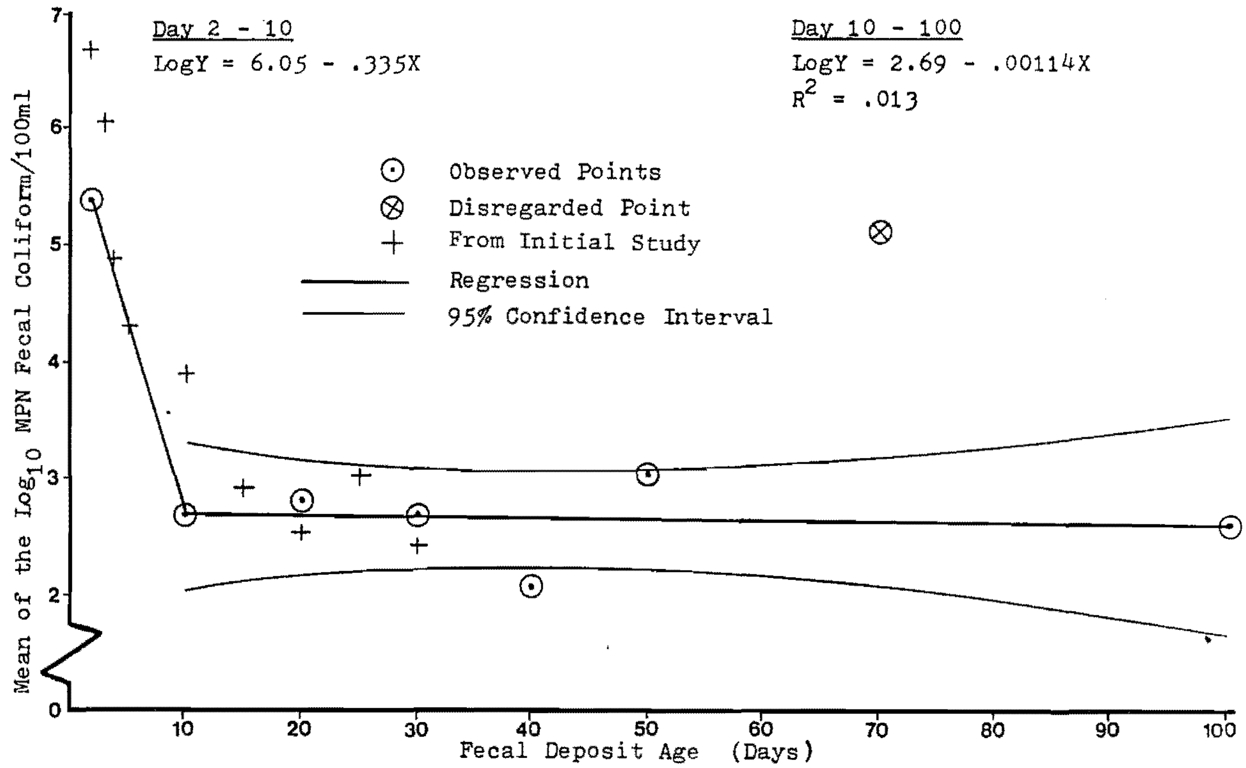

Figure 9. Two log-normal regressions were determined from the 5 minute, fecal coliform release from standard cowpies. Five minute data from the initial study are also shown.

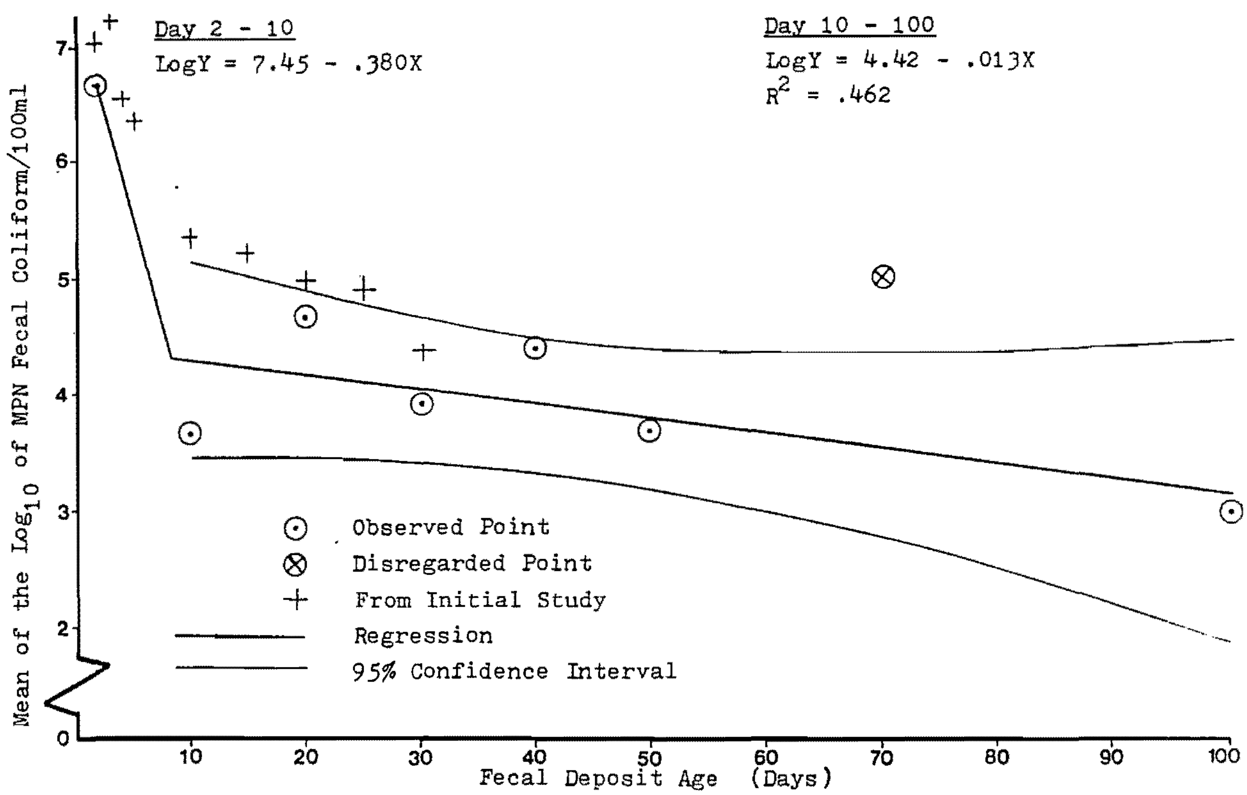

Figure 10. Two log-normal regressions were determined from the 10 minute, fecal coliform release from standard cowpies. Ten minute data from the initial study are also shown.

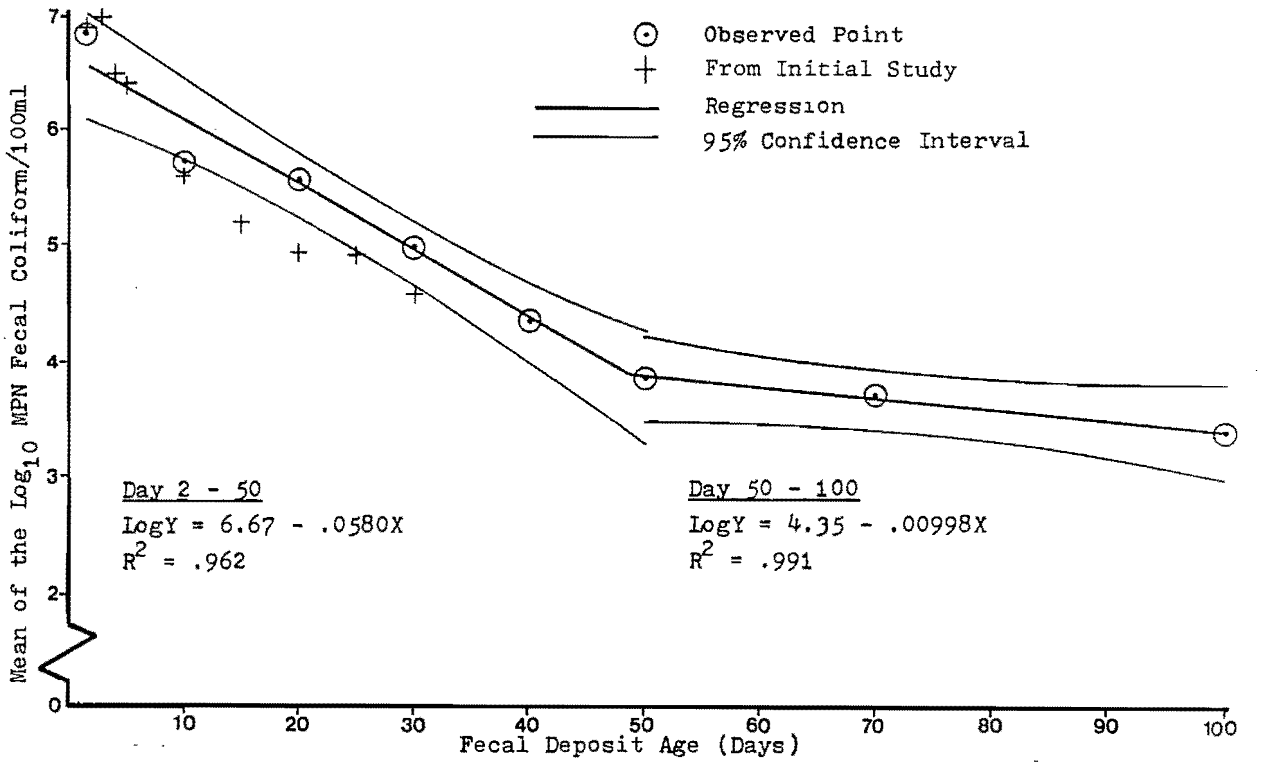

Figure 11. Two log-normal regressions were determined from the 15 minute, fecal coliform release from standard cowpies. Fifteen minute data from the initial study are also shown.

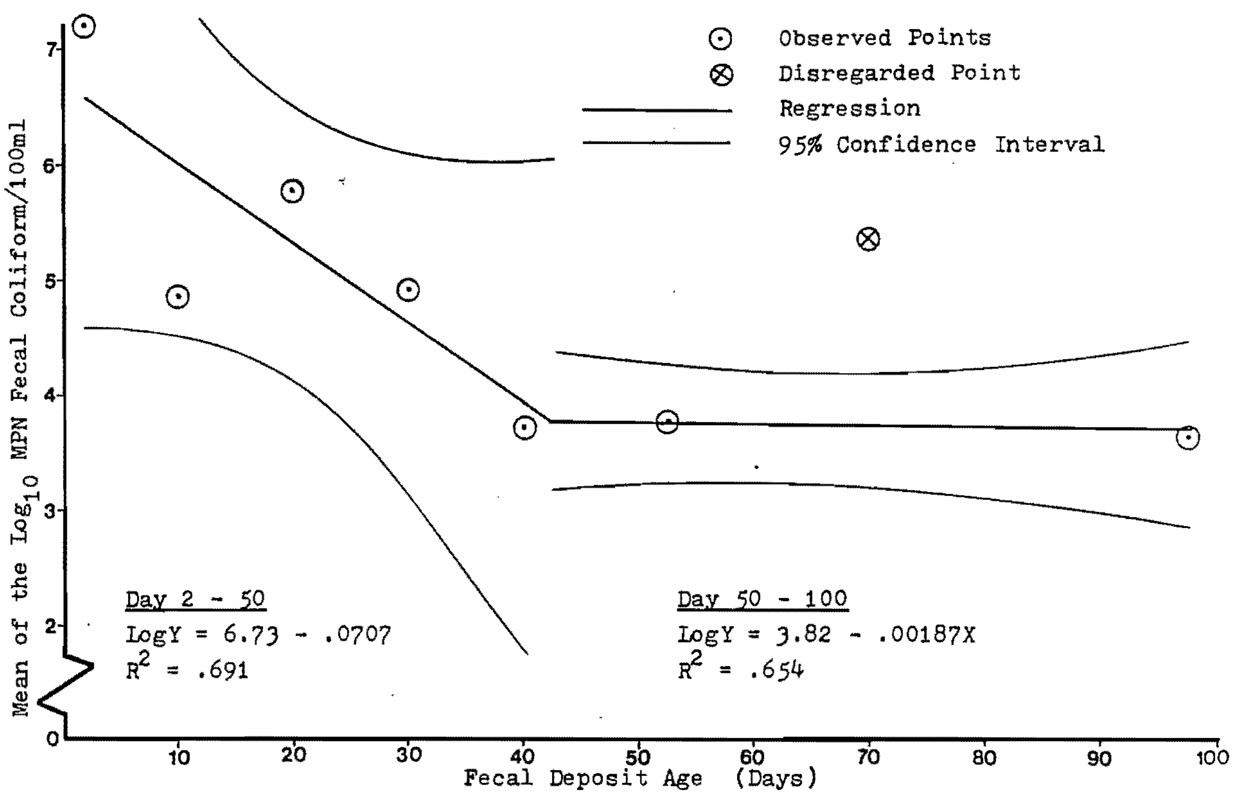

Figure 12. Two log-normal regressions were determined from the 20 minute, fecal coliform release from standard cowpies.

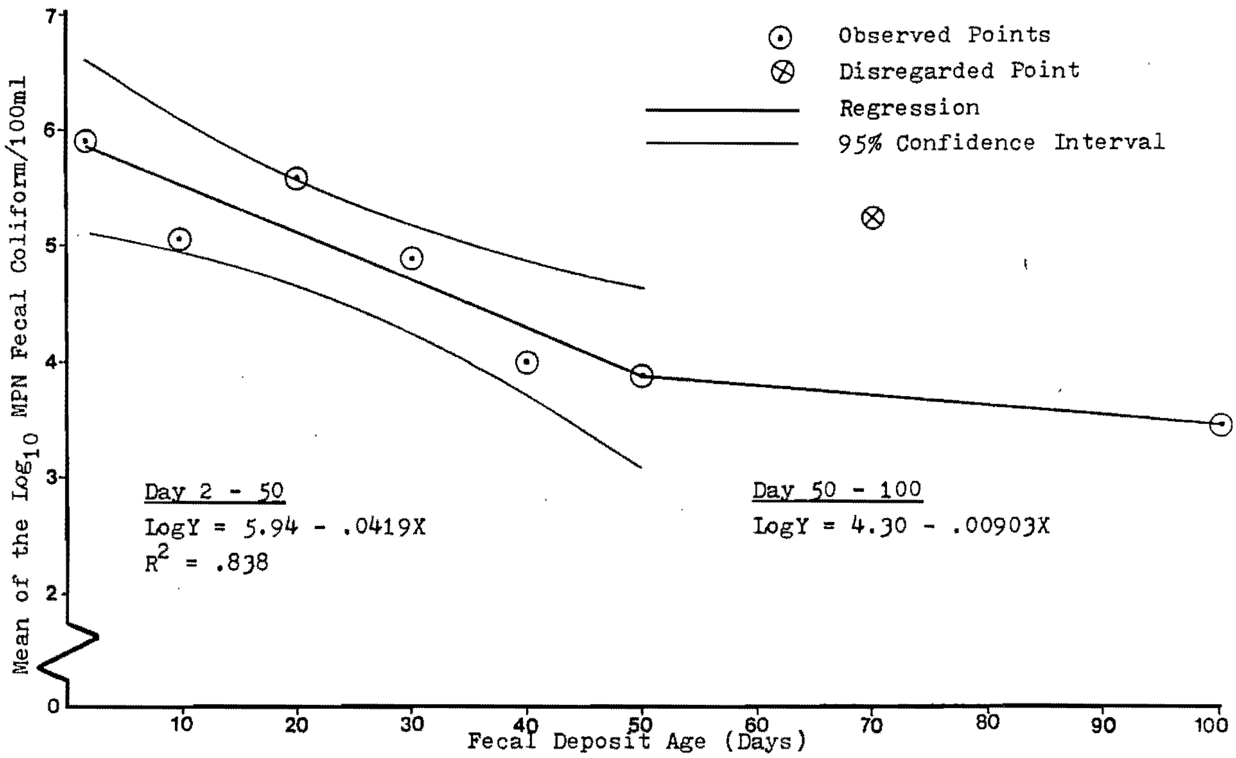

Figure 13. Two log-normal regressions were determined from the 25 minute, fecal coliform release from standard cowpies.

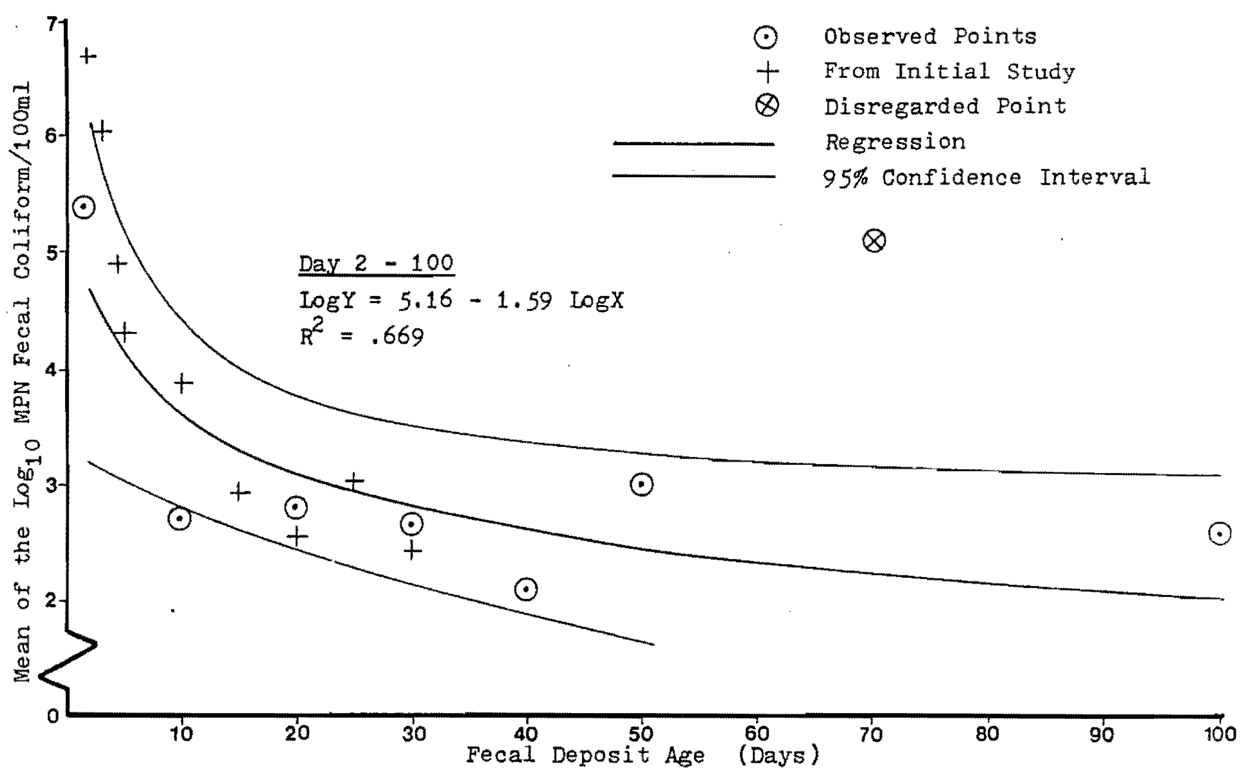

Figure 14. The 100 day log-log regression determined from the 5 minute, fecal coliform release from standard cowpies. Five minute data from the initial study are also shown.

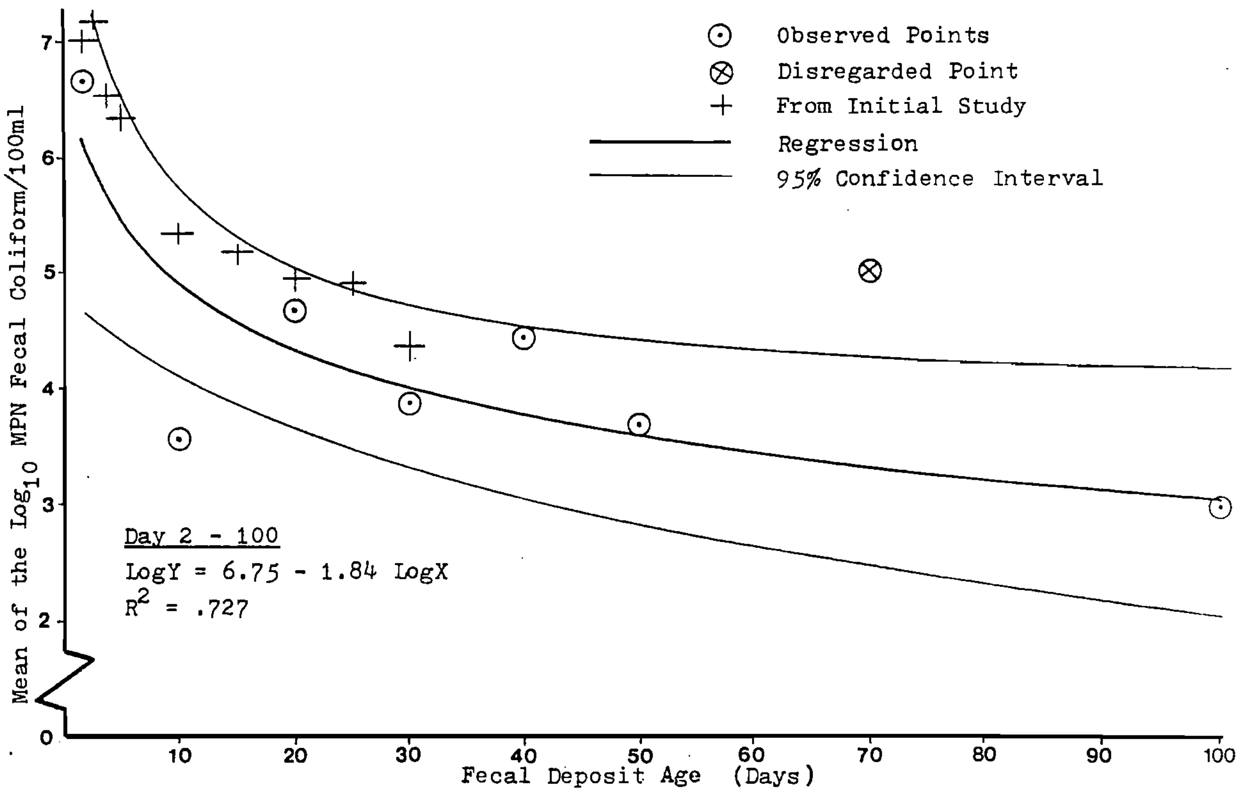

Figure 15. The 100 day log-log regression determined from the 10 minute, fecal coliform release from standard cowpies. Ten minute data from the initial study are also shown.

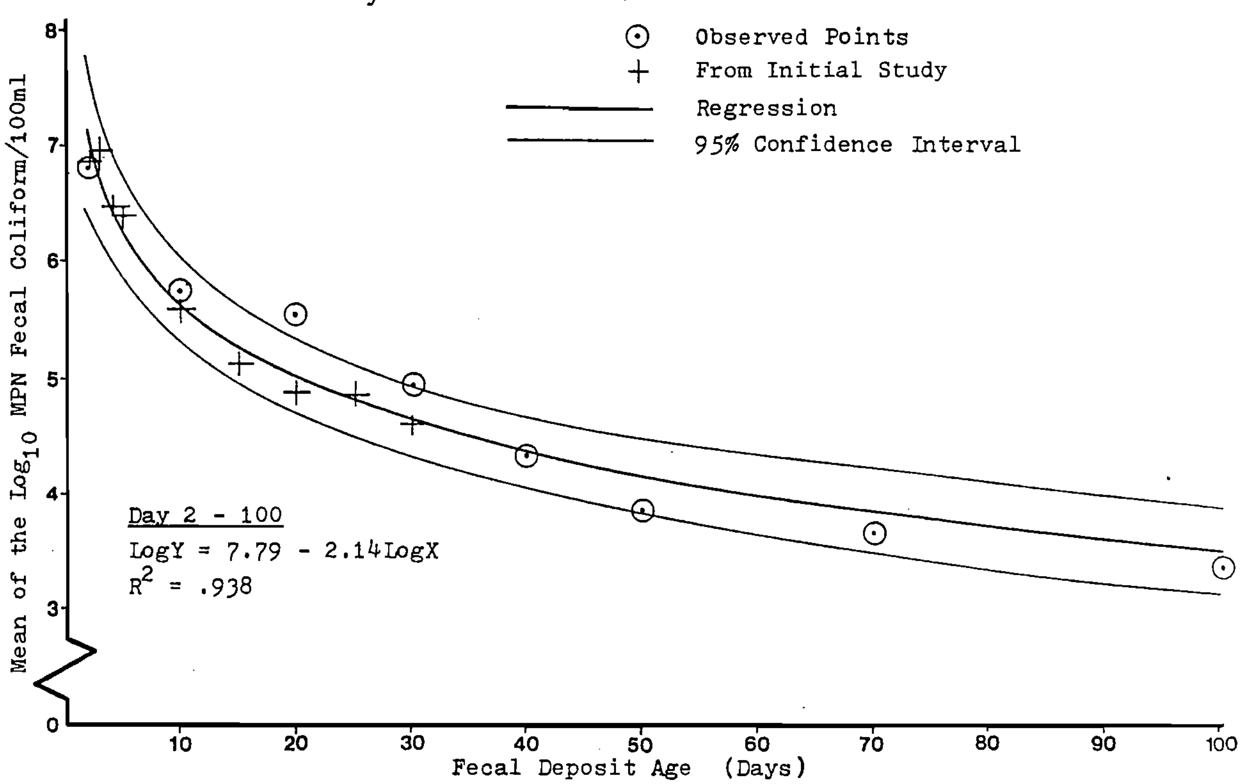

Figure 16. The 100 day log-log regression determined from the 15 minute, fecal coliform release from standard cowpies. Fifteen minute data from the initial study are also shown.

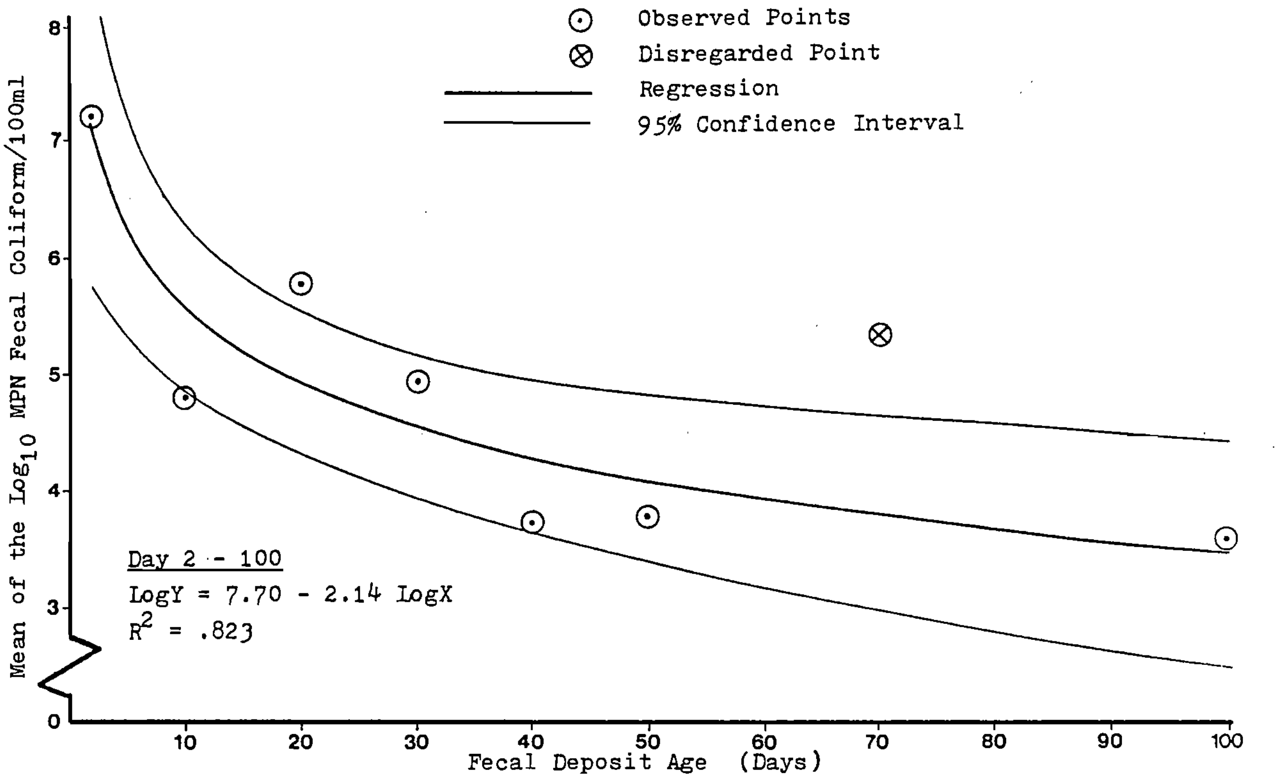

Figure 17. The 100 day log-log regression determined from the 20 minute, fecal coliform release from standard cowpies.

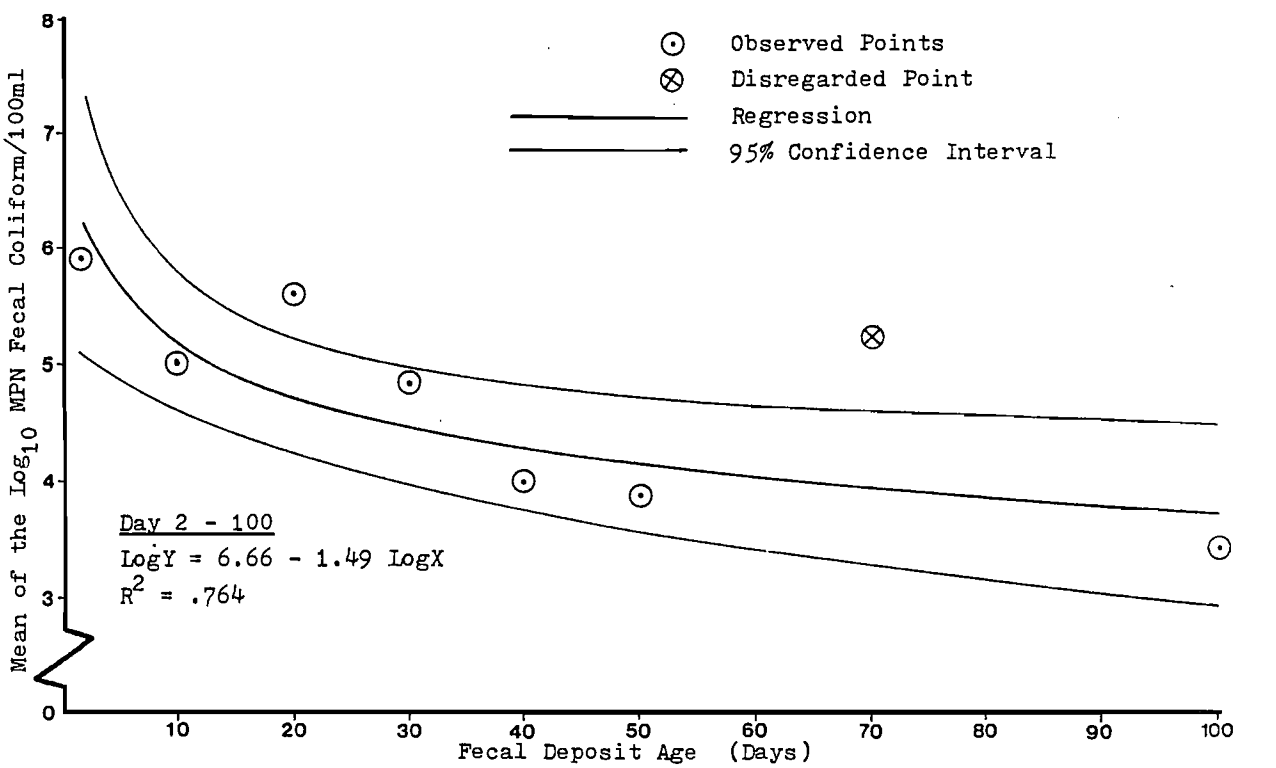

Figure 18. The 100 day log-log regression determined from the 25 minute, fecal coliform release from standard cowpies.

Table 6. Peak fecal coliform counts (transformed to the $\log_{10}$).

Age Days	Mean FC/100 ml	Variance	N
Standard Cowpie Treatment			
2	6.69	0.155	9
10	5.73	0.146	9
20	5.53	0.082	9
30	4.95	0.398	9
40	4.42	0.328	9
50	3.84	0.492	9
Recurrent Rainfall Treatment			
2	6.69	0.155	9
10	5.31	0.419	9
20	3.64	0.167	9
30	3.46	0.315	9
40	2.49	0.180	9
50	3.26	0.628	9
Natural Dung-pile Treatment			
2	7.18	0.239	9
10	6.00	0.718	9
20	5.09	0.320	9
30	4.73	0.564	9
40	4.99	0.763	9
50	5.18	0.244	9
Low-intensity (22.9 mm/hr) Rainfall Treatment			
2	6.62	0.074	9
10	5.84	0.189	9
20	6.38	0.217	3
Mid-intensity (50.8 mm/hr) Rainfall Treatment			
2	6.69	0.155	9
10	5.73	0.146	9
20	5.53	0.082	9
High-intensity (68.6 mm/hr) Rainfall Treatment			
2	6.75	0.126	9
10	5.54	0.207	9
20	4.70	0.454	9

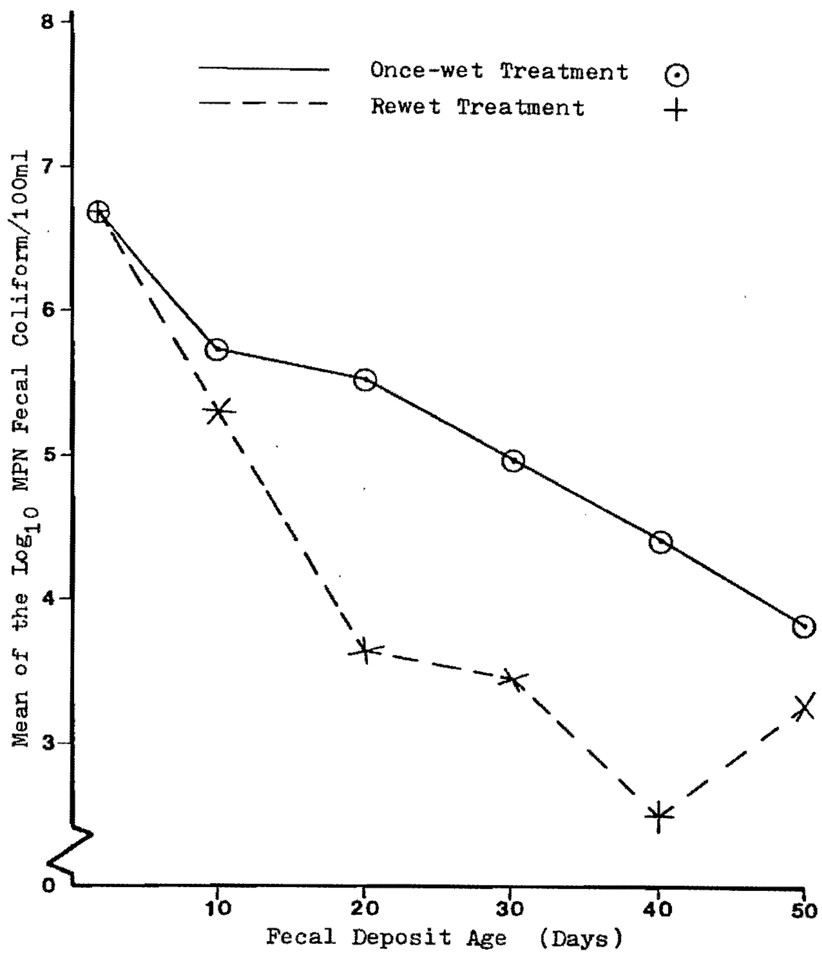

Figure 19. Fecal coliform release from once-wet cowpies compared to the fecal coliform release from rewet cowpies.

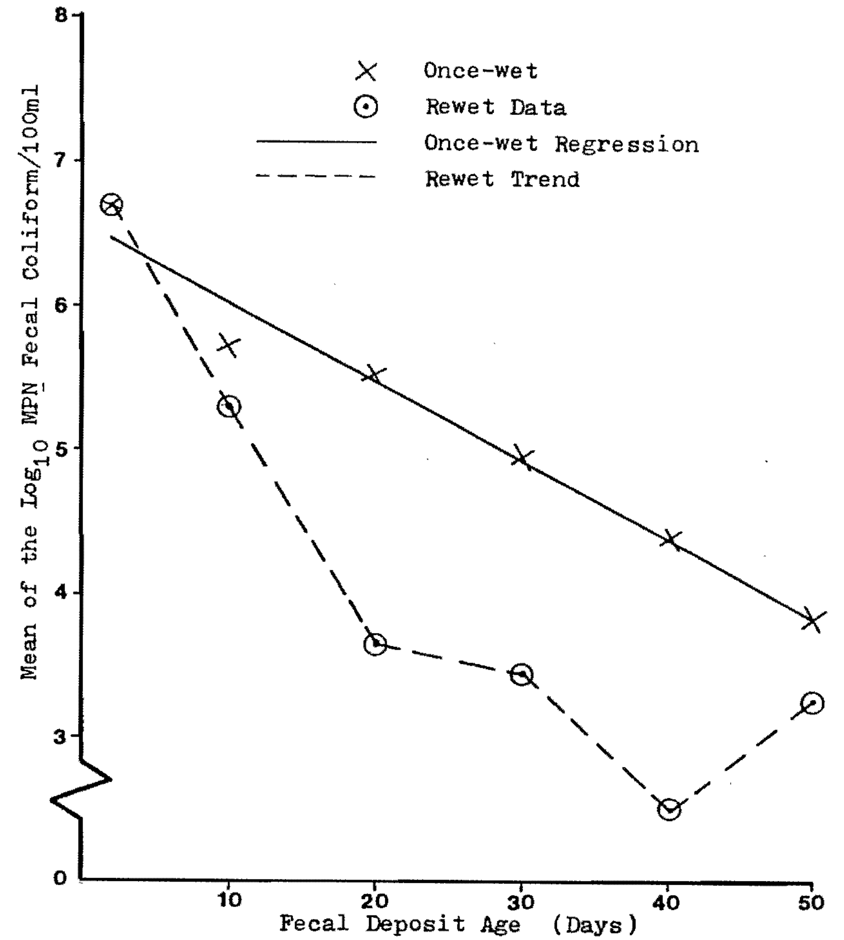

Figure 20. The peak fecal coliform release from rewet cowpies compared to the log-normal regression for the once-wet fecal deposits.

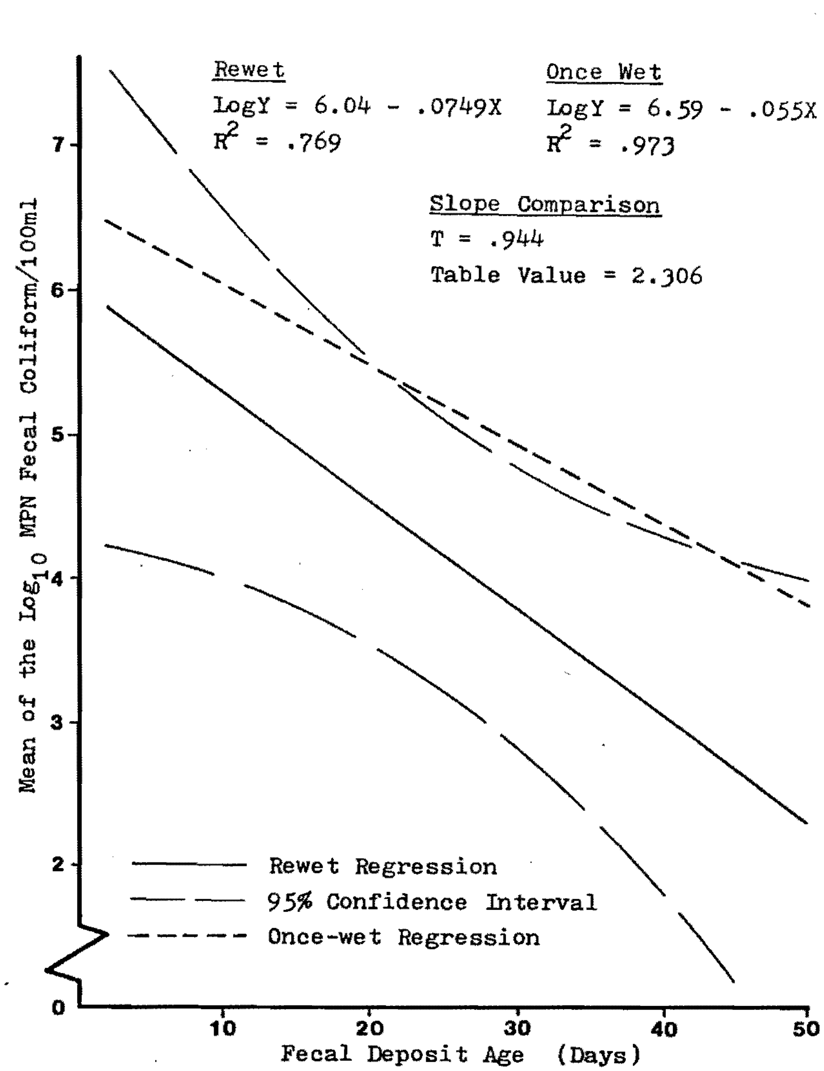

Figure 21. A log-normal regression was determined from the peak fecal coliform release from rewet fecal deposits and compared to the log-normal regression for the once-wet fecal deposits.

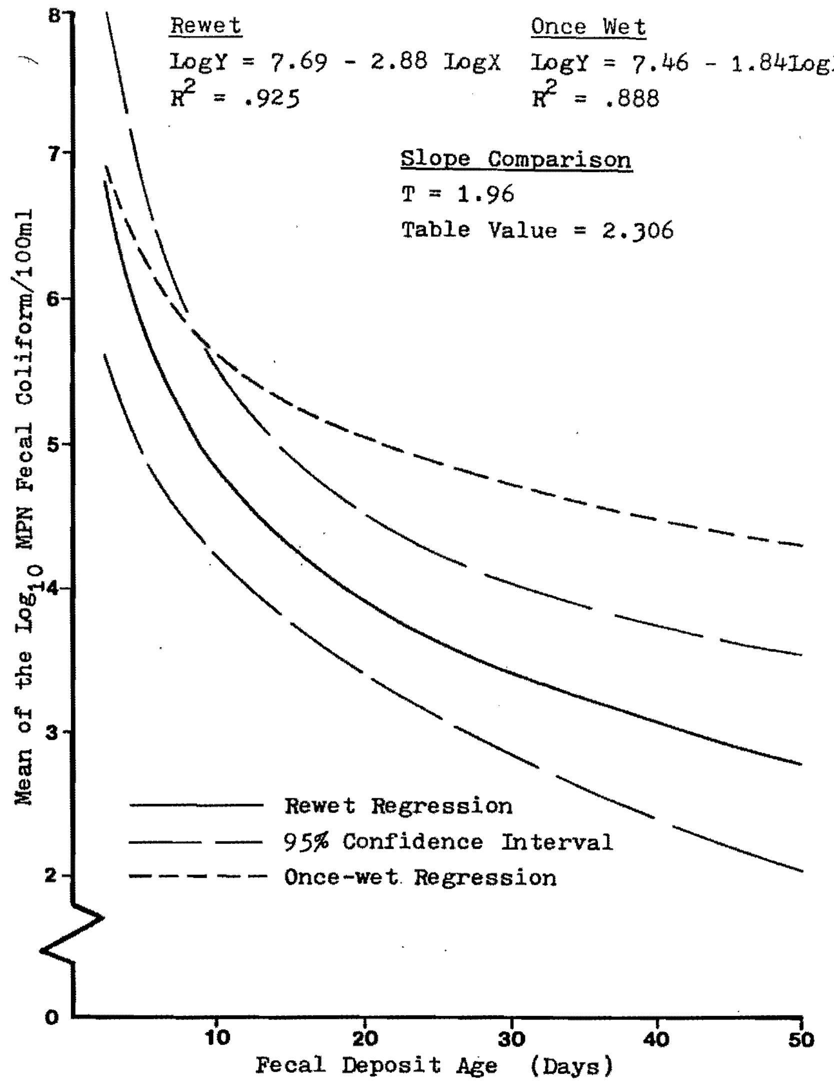

Figure 22. A log-log regression was determined from the peak fecal coliform release from rewet fecal deposits and compared to the log-log regression for the once-wet fecal deposits.

The log-log, rewet regression had an R^2 of 0.925, and its slope was also not significantly different from the slope of the once-wet regression (Figure 22). The once-wet regression line fell largely outside of the confidence limits of the rewet regression for both the log-normal and the log-log analyses (Figures 21 and 22).

Comparison of Cowpie and Dung Pile Treatments

The mean, fresh weight of naturally-occurring fecal deposits was 1.24 kilograms. Natural dung piles had a fresh weight of 1.24 $\pm$ 0.11 kilograms; this was 37.5 percent more than standard cowpies.

The FC release from natural dung piles was higher than the release from cowpies at ages 2, 10, 40, and 50 days, and it was lower at ages 20 and 30 days. These differences were significant only at ages 2 and 50 days. The log-log regressions of the natural dung-pile treatment and the standard cowpie treatment were not significantly different from each other; their slopes were not significantly different, and each regression fell entirely within the other's confidence limits (Figure 23).

Effect of Rainfall Intensity

There were no significant differences among the three intensity treatments at days 2 and 10, but there were significant differences at day 20 (Table 6). At day 20, the low-intensity treatment had the highest peak FC release, and the high-intensity treatment had the lowest peak FC release (Figure 24). For cowpies that had aged 20 days, the high-intensity rain produced a peak release at 10 minutes; the mid-intensity rain peaked at 15 minutes; and the low-intensity rain did not peak before 25 minutes (Figure 25). Release rates with the 20 day low-intensity treatment remained below 300 FC per 100 milliliters for the first 15 minutes, and the count rose sharply at 20 minutes (Figure 25). The 2 day and 10 day intensity treatments responded much earlier.

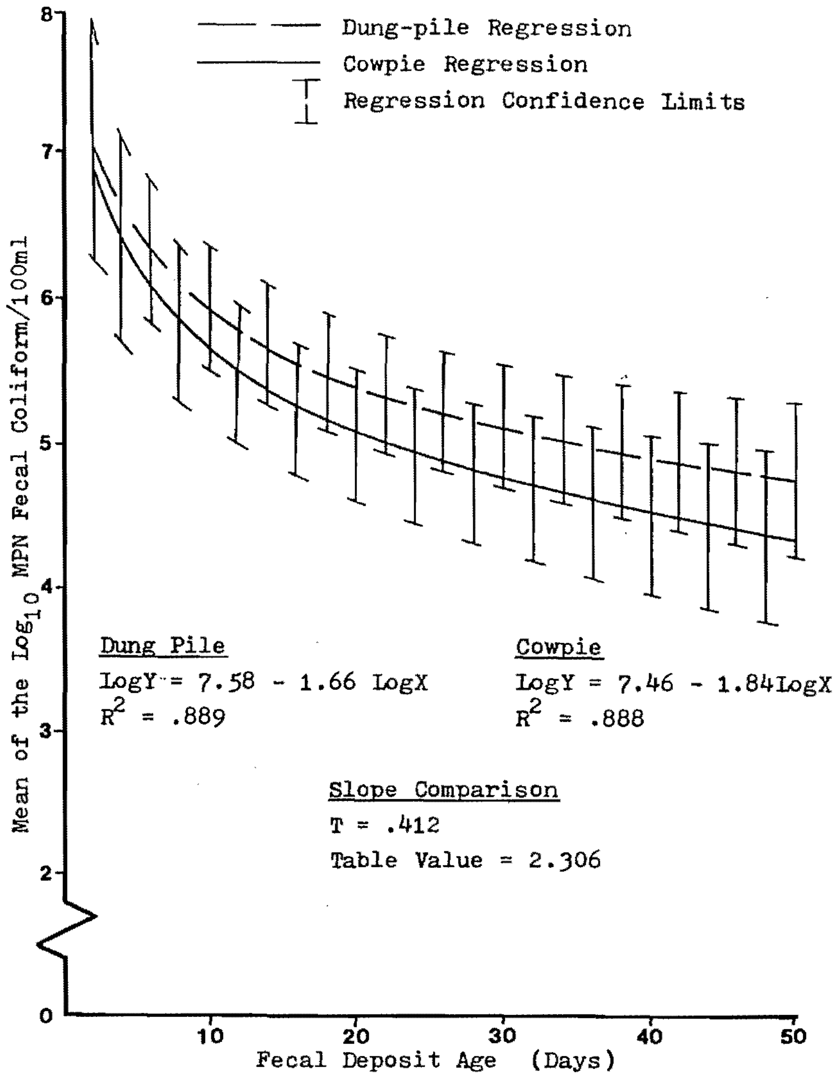

Figure 23. The log-log regression for natural dung piles compared to the standard cowpie regression.

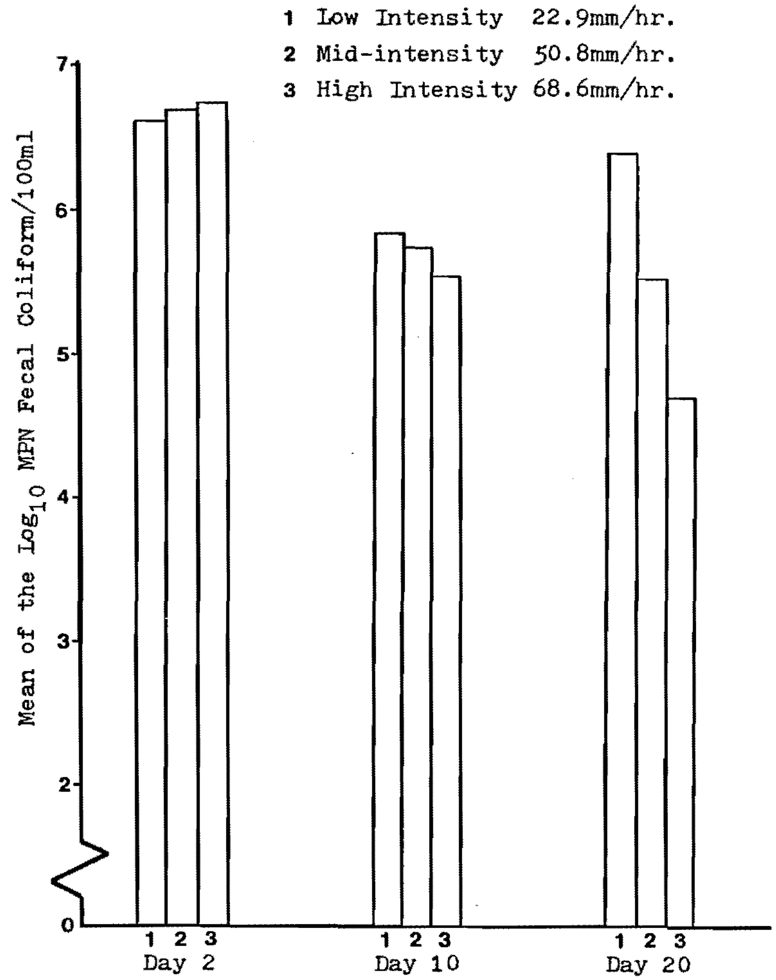

Figure 24. Peak fecal coliform counts from standard cowpies at three simulated rainfall intensities.

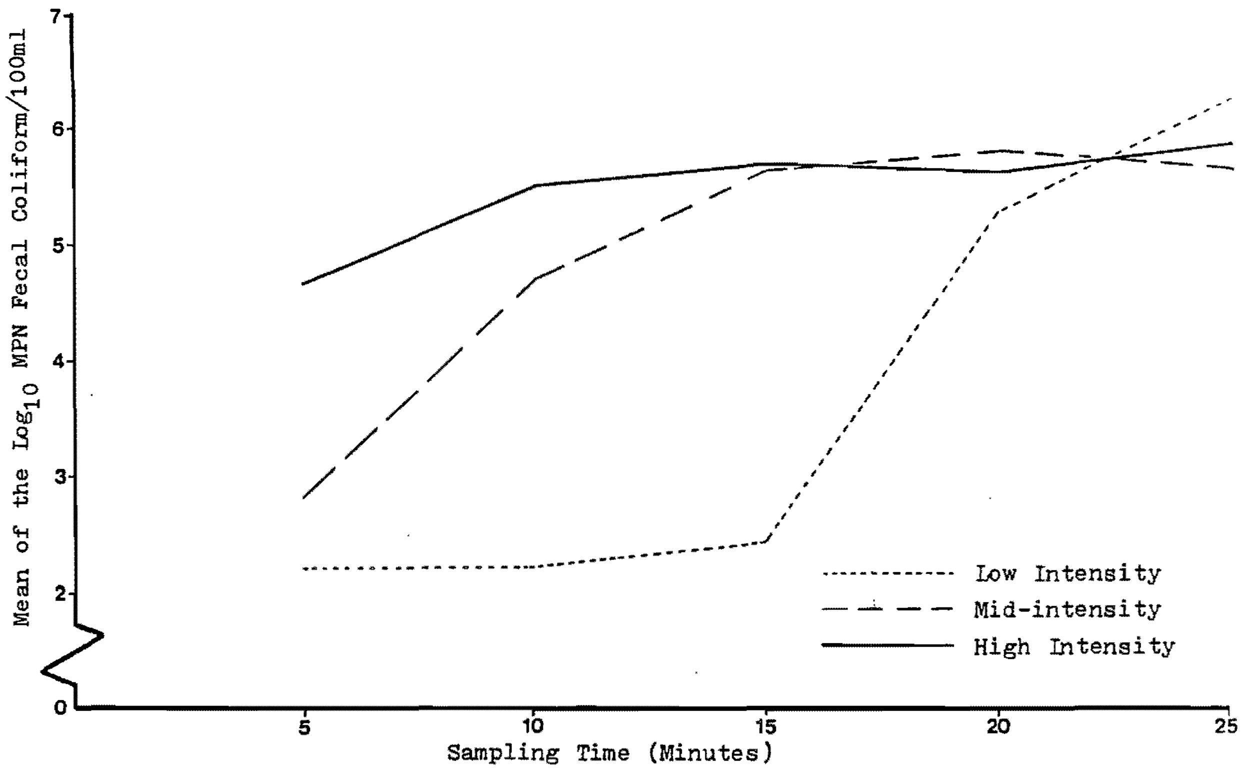

Figure 25. Fecal coliform counts at 5 minute intervals at three rainfall intensities (22.9, 50.8, and 68.6 mm/hr) using 20-day-old cowpies.

DISCUSSION

Public Health Significance of Fecal Coliform

Initial Studies

Interpretation of the regression curves for the 10, 15, and 10 and 15 minute pooled data (Figures 4, 5, and 6) is somewhat clarified by considering the generalized growth curve of a bacterial culture (Figure 26). Such a comparison suggests that the regression curves are just the second half of a growth curve starting with the retardation phase.

Apparently, the fecal coliform population leaves the bovine system while in the exponential phase. This situation is made possible by an average of 12 defecations per day, thus resulting in a continuous input and output situation. Once voided from the body, the fecal coliform population enters the retardation phase in which the growth rate begins a decline that continues until growth ceases (Lynch and Poole 1979). In the situation studied, this decline in growth rate is quite rapid, with growth apparently ceasing shortly after being voided. The progressive decline of the specific growth rate is due, in this particular case, to the depleting growth resource. That growth does continue once the bacteria are outside the host should not be too surprising since it is known that fecal coliforms can survive and even multiply under such adverse conditions as dilute nutrient-low temperature environments (Hendricks and Morrison 1967).

From the retardation phase, the bacterial population moves into a maximum population phase (sometimes referred to as the stationary phase). Once in this phase, the population remains metabolically active even though active growth has ceased. The potential for continued growth is retained should favorable growth conditions be established (Lynch and Poole 1979).

According to Stanier et al. (1979), cells in the maximum population phase are small relative to cells in the exponential phase since during the retardation phase cell division continued for a period after the increase in mass had stopped. Along with this

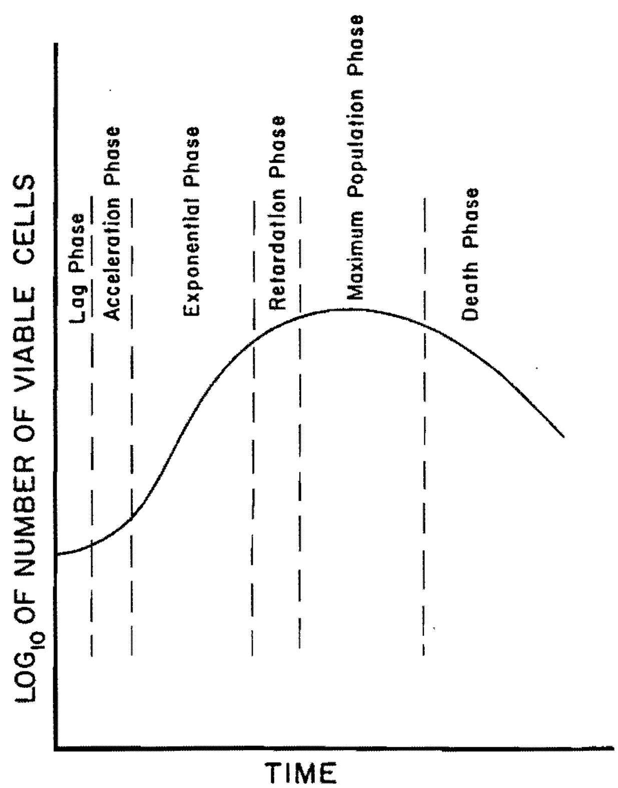

Figure 26. Generalized growth curve of a bacterial culture. Modified from Lynch and Poole (1979).

size differential, the cells are more resistant to adverse physical (heat, cold, radiation) and chemical agents.

These facts would help explain two things about the results obtained in this study. One is the duration of the maximum population phase. According to the curves derived for this study, this phase lasts approximately 2 days. More importantly, though, these facts help explain the large numbers of fecal coliforms being released from fecal deposits at 30 days of unrained-on age. Even though there is a drop of more than two orders of magnitude between 2 days and 30 days, samples taken at 10 and 15 minutes from the 30 day test produce counts per 100 ml on the order of 40,000. This indicates that a large population of fecal coliforms still exists in a fecal deposit long after the deposit has been thoroughly dried.

However, it is clear that the capacity to survive under these conditions is limited with the eventual result being the onset of cell death and lysis (Lynch and Poole 1979). Death results from a number of factors with an important one being depletion of the cellular reserves of energy. Like growth, death is an exponential function (Stanier et al. 1979).

The 5 minute samples show only an exponential death phase. It is felt that because of the increasing time to an equilibrium output of fecal coliforms with increasing unrained-on age, the growth and leveling off periods of the first 3 days do not show up in the regression analysis. A cubic equation would show the occurrence of the retardation and maximum population phases. Indeed, the cubic equation derived from regression analysis produced a higher R^2 value than did the linear equation. However, as stated previously, the cubic equation was not significantly better than the linear equation.

Regression analysis was performed on the 10, 15, and 10 and 15 minute pooled data using only data from 3 days on. This was done to see if the death phases for these sampling times were linear and similar in nature to the death phase described by the 5 minute analysis. In all three cases, the death phase is indeed a linear relationship though not producing as steep a decline as did the 5 minute analysis. Figures 4 5, and 6 give the linear equations derived and their corresponding goodness of fit statistics.

The fecal deposit appears to act as a protective medium for the bacteria within. The deposit surface was dried quite hard after only 2 days. At 15 days, a deposit broken open was essentially completely dry. This rapid dessication during the retardation and maximum population phases coupled with the increased resistance of the cells during the former phase is responsible for a large bacterial population, even after 30 days of drying. As suggested previously by Buckhouse and Gifford (1976), bacteria under such conditions may be viable for as long as several years.

It should be remembered, however, that fecal coliforms are indicators of bacterial contamination that do not themselves threaten health. To assess the hazard, fecal coliform counts must be related to the presence of pathogenic organisms in the same environment. This has been done for the stream environment where Geldreich (1970) found that when fecal coliform counts were greater than 2,000 per 100 ml, Salmonella isolations should occur with near 100 percent frequency. If this rule can be applied to the runoff samples collected during this study, even after 30 days of drying, a fecal deposit is capable of releasing pathogenic organisms when rained on for as little as 10 minutes.

The above conclusion assumes that Salmonella survive within a fecal deposit about as long as do fecal coliforms. Death rates of bacteria are dependent on the environment (which, in this case, are the same for both types of organisms) as well as on the particular organism itself (Lynch and Poole 1979, Stanier et al. 1979). That a similarity does indeed exist does not seem too unlikely as it is known that salmonellae can survive in water for lengths of time similar to those enjoyed by fecal coliforms in water (McFeters et al. 1974). Therefore, it seems reasonable to assume that not only are cattle fecal deposits potent suppliers of fecal coliforms even after 30 days of drying in intense summer sunlight, but that they are also a potential source of pathogenic bacteria.

Phase II Studies

Fecal deposits that are 100 days old are still a potential source of FC; their release exceeds recreation water quality standards. However, it would require the FC release from approximately 1,000, 100-day-old fecal deposits to equal the release from one, 2-day-old fecal deposit. The contribution of 100-day-old feces may be relatively minor when fresh feces are being deposited.

The age treatment was analyzed using log-normal and log-log regressions, and both regressions fit the data about equally well. Other data were therefore used to determine which regression is more appropriate. Data from the initial study definitely fit the log-log regression better than it fit the log-normal regression; this suggests that the log-log regression may be preferable. The data from the natural dung-pile treatment also fit the log-log regression better.

The 5 minute and 10 minute log-normal regressions showed a distinctly different FC decline pattern from the 15, 20, and 25 minute log-normal regressions (Figures 9-13). This difference appears to have resulted from a delay in reaching peak FC counts as the fecal deposits dried out. The fecal deposits appeared to have dried out some time between 10 and 20 days.

Effect of Recurrent Rainfall

For subsequent rainfalls, the peak FC counts were considerably lower than what they were for the first rain (Figure 19). This may be due to a significant number of FC being leached out of the fecal deposits during previous wettings. Geldreich (1966) estimated that the average gram of fresh cow feces contains about 2.3×10^4 FC; this would mean that a fresh, standard cowpie should contain about 2.1×10^8 FC. The 2-day-old, standard cowpies were calculated to have released 2.16×10^8 FC at the end of the 25 minute rainfall. Although the above average is only an approximation, the loss of 2.16×10^8 FC would remove a large share of the coliforms from the cowpie. The peak FC counts declined by the third wetting to a point where it would require 300, 20-day-old rewet fecal deposits to equal the release from one, 2-day-old fecal deposit.

FC release with recurrent rainfall is variable. Several factors may be relevant. Wetting and subsequent drying of fecal deposits may affect the dieoff rate of FC. Wetting may induce FC growth in the fecal deposits. Prior wetting may change the infiltration and erosion patterns in the fecal deposits. Further study of rewet fecal deposits is necessary before their release behavior can be adequately predicted or explained.

Comparison of Dung Pile and Cowpie Treatments

The regression for the standard cowpie was not significantly different from that for the natural dung-pile regression (Figure 23). This is not surprising if FC release is a function of the initial FC numbers in the fecal deposit. The initial FC numbers are proportional to the fecal-deposit weight (Geldreich 1966). The natural dung piles weighed an average of 37.5 percent more, fresh weight, than a standard cowpie, so the dung piles initially contained 37.5 percent more FC than the standard cowpies. A 37.5 percent difference is difficult to detect when the nontransformed coefficient of variation was as high as 220 percent and never fell below 65 percent.

The real question is whether the fecal-deposit shape and volume significantly affect the FC release rate. Since the standard cowpie and the dung-pile regressions are not significantly different, the standard cowpie regression can be used to determine FC release from naturally-occurring fecal deposits.

Effect of Rainfall Intensity

Rainfall intensity did not significantly alter fecal coliform leaching at days 2 or 10; ages when standard cowpies were still partially moist. At day 20, when the standard cowpies were completely dry, the effect of rainfall intensity was significant. The volume of rainfall after 15 minutes of low-intensity rain equaled the volume of rainfall after 5 minutes of high-intensity rain. Yet the 20 day, high-intensity treatment showed significant response at 5 minutes while the 20 day, low-intensity treatment showed little response at 15 minutes. This low-intensity, delayed response suggests that rainfall intensity can affect the flow path through a dry fecal deposit.

PART II

PRELIMINARY NONPOINT SOURCE TRANSPORT MODEL

METHODS AND PROCEDURES

The development of a generally applicable model was based on physical equations for overland flow and coliform movement as described in detail below. Verification data for these equations were collected from controlled and "natural" surfaces during simulated rainfall events.

Experimental Site

Both the control and "natural" studies were done at the USU Ecology Center Compound located at the mouth of Green Canyon in North Logan. The availability of North Logan drinking water and close proximity to laboratory facilities made the compound ideal for this study.

Control Surface

To provide uniform flow hydraulics, a surface with constant characteristics was required. Therefore, a concrete plot, 30.48 m (100 ft) long by 1.83 m (6.0 ft) wide was constructed on a 6 percent slope (Figure 27). Since the bacteria adsorb to interfaces, the runoff surface was given a smooth trowel finish to minimize roughness, hence, adsorption sites.

Water was collected from the rectangular runoff surface and routed into a 0.15 m (0.50 ft) trough through a triangular transition at the lower end of the plot (Figure 27). Runoff was measured by a 0.15 m (0.50 ft) HS flume (Brakensiek et al. 1979) which was equipped with a Stevens Type F water level recorder that was modified so that the pen would cross the chart in 30 minutes. During simulated runoff events, both the transition section and the flume were covered so that they did not contribute to runoff.

Natural Surface

After collecting the needed data from the concrete surface, sides were constructed for the concrete runoff plot, and soil to a depth of 61 cm (24 in) was added (Figure 28). The soil was overburden from a gravel pit near Millville and had high clay (40 percent) and silt (40 percent) contents. The soil was placed into the box in October 1980, and allowed to settle over the winter.

The triangular transition from the soil surface to the flume was constructed from sheet metal. The slope of the transition was steep, approximately 40 percent; and during runs, it was covered. The same trough arrangement was used as in the controlled studies.

Rainfall Simulator

The rainfall simulator was based on the Colorado State University design (described by Smith 1979) as modified by Lusby and Toy (1976). The design utilized Rainjet 78-C sprinkler heads on 1.90 cm (0.75 in) diameter risers located 3.05 m (10 ft) above the ground. The sprinklers were spaced on

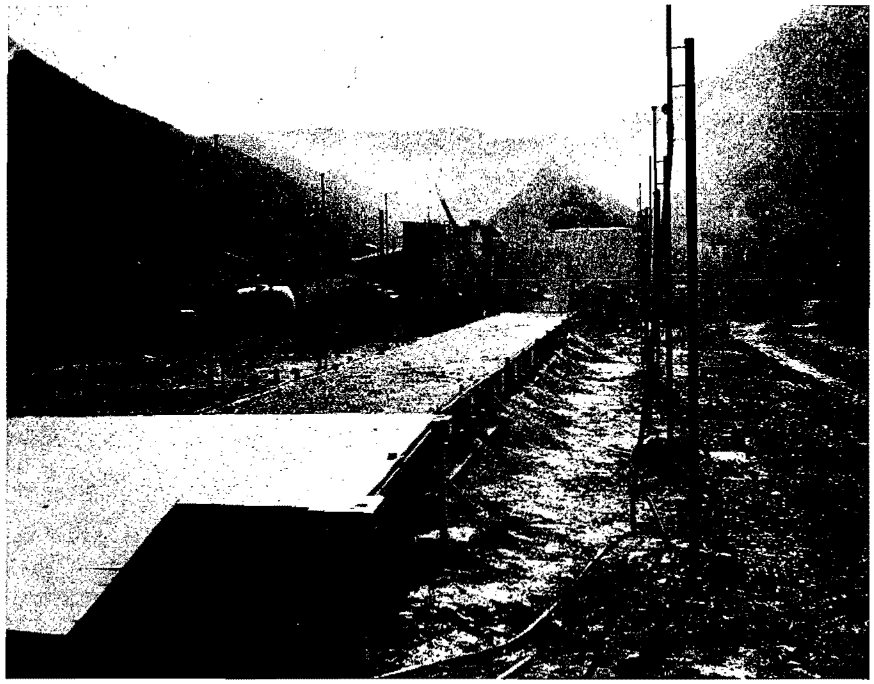

Figure 27. Concrete runoff surface during a simulation.

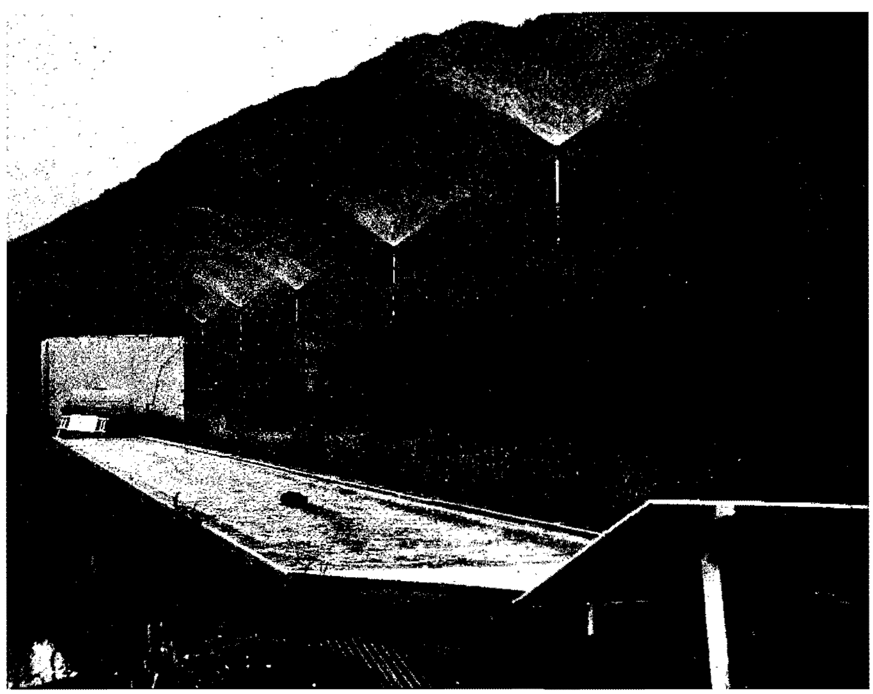

Figure 28. Soil surface during a simulation.

6.10 m (20 ft) equilateral triangles. To provide uniform coverage of the runoff plot, two rows and a total of 11 sprinklers were required. A 10 h.p. pump supplied water to the system.

Calibration of the system revealed that the maximum rainfall intensity obtained was 3.8 cm/hr (1.5 in/hr); Lusby and Toy (1976) operated their system at 5.08 cm/hr (2 in/hr). They had 5.08 cm (2 in) feeder lines going to their risers, and the 2.54 cm (1 in) lines used in this study could not provide the capacity needed to attain the higher intensity.

System operating pressure affects drop size distribution. Lusby and Toy (1976) operated their system at a constant pressure of 0.19 N/m^2 (28 psi) at a height of 0.46 m (1.5 ft) above the feeder line. This system operated at a pressure of 0.14 N/m^2 (20 psi) at 2.44 m (8 ft) above the ground. Solution of the Bernoulli equation indicates a loss of approximately 0.02 N/m^2 (2.7 psi) for 1.83 m (6 ft) elevation, which means that Lusby and Toy (1976) had a pressure equivalent to 0.17 N/m^2 (25.30 psi) at 2.44 m (8 ft) above the ground. Data obtained from Neff[1] indicated that, for riser pressures greater than 0.14 N/m^2 (20 psi), the D_{50} drop size levels out very quickly and is essentially constant. With higher pressure, the D_{75} drop size becomes smaller, and the drop size distribution shifts to the lower sizes. The data provided by Neff indicated little difference in the drop size distributions between 0.14 N/m^2 (20 psi) and 0.17 N/m^2 (25 psi), but differences were noted between these pressures and 0.21 N/m^2 (30 psi) over the upper 30-40 percent of the distribution. The kinetic energy and hence the erosive power of the rainfall was shown by Neff to decrease with increasing pressure. This decrease in kinetic energy was attributed to the smaller D_{75} drop size since the larger drops contribute more kinetic energy (Foster et al. 1981). Given these data, it was concluded that the system performed adequately at the pressures utilized.

The water supply for the sprinkler system used to simulate rainfall on the controlled surface during the first field season was a 1.89 m^3 (500 gal) plastic tank. To keep bacteria from growing in the tank, it was periodically cleaned with chlorine and flushed. To prevent any residual chlorine that may have been present in the water from affecting the coliforms, a 10 percent solution of sodium thiosulfate was added to the water for dechlorination as prescribed in Standard Methods (American Public Health Association 1975). The following season, a 3.78 m^3 (1000 gal) metal tank was linked to and gravity fed from the plastic tank. Prior to installation, this tank was steam cleaned and flushed. The dechlorination of the water followed the same procedure as the previous season.

The plastic tank provided enough water to simulate a 3.8 cm/hr (1.5 in/hr) storm for 5 minutes. For the concrete surface, this was more than enough, since the equilibrium time until rainfall excess equals runoff was approximately 180 sec. The addition of the 3.78 m^3 (1000 gal) tank increased the run time to 20 min. Given the increased roughness and the occurrence of infiltration, the equilibrium time could not be calculated for the soil surface, but it was evidently longer than 20 minutes because equilibrium hydrographs were not observed during this phase of the study.

Study Design

The first part of this study deals with the movement of indicator bacteria from the source material into overland flow. For this model, the concentration

[1]Personal communication, Earl Neff, Res. Hydraul. Eng., USDA-ARS, Sidney, Montana.

of fecal coliforms at a point immediately downslope becomes a point source from which the concentration is routed overland. Because bacteria attach themselves to interfaces, for example, the solid surface-water interface, the length of overland flow was chosen as the primary variable to use in the model. Fresh fecal material, obtained from control cattle at the USDA Poisonous Plant Laboratory, was placed at various distances above the outlet, and runoff concentration were measured over time. The distances utilized with the concrete control surface were 1.52 m, 3.05 m, 6.10 m, 9.14 m, 12.19 m, 15.24 m, 22.86 m, 27.43 m, and 30.48 m (5, 10, 20, 30, 40, 50, 75, 90, and 100 ft. respectively). Prior to every run, the surface was scrubbed with a chlorine solution to remove any bacteria, and another simulation was started after the surface dried. Samples for bacterial analysis were collected at the slope outlet at 30 sec intervals up to the time of equilibrium. Also, there was no runoff for the first 60 sec of simulation; therefore, the 90 sec sample was the first collected. After the 180 sec equilibrium time, samples were collected at 240 sec and 300 sec, and then the rainfall was shut off.

On the soil surface, the simulations lasted 20 min, and samples were taken every 5 min. There was a 24 hour period between runs during which the runoff trough and flume were cleaned with a chlorox solution. Runoff did not start until 8 min after rainfall started, which meant samples were taken at 10, 15, and 20 min. Two control runs were made prior to placing any fresh fecal material on the surface to collect data on background counts. Source distances for the simulation were 1.52 m, 3.05 m, and 15.24 m (5, 10, and 50 ft, respectively).

Bacteriological Analyses

Samples of runoff were collected in sterile Whirlpak bags and brought to the laboratory for analysis within 2 hr of collection. Samples were kept in a styrofoam cooler until analysis, but they were not put on ice because of the short time period between collection and analysis.

All samples were analyzed for total and fecal coliforms using the multiple tube methods (MPN) described in Standard Works (APHA 1975). Even though the MPN is a probabilistic method with low precision, it was preferred over the membrane filter (MF) technique because of its ability to handle turbid samples. Samples with high turbidity may clog the pores of a membrane filter resulting in either no growth of colonies, since they are not in contact with the medium, or rapid spreading of colonies, making counting difficult. If counts are high enough, filtration may be used with the MF technique, but for samples with low counts and high turbidity, the MPN method is preferred (United States Environmental Protection Agency, USEPA 1978). For the initial phase of this project conducted on a concrete surface, MF could have been utilized; but for data from soil surfaces, MPN methods would be required. To maintain continuity, it was decided to use the MPN method throughout the project so that the results could be compared.

The MPN method is composed of three parts, each designed to collaborate an estimate of the number of all aerobic and facultative anaerobic, gram-negative, nonspore forming, rod shaped bacteria that ferment lactose at 35°C. In the first part of the presumptive test, in which lauryl sulfate tryptose broth (LST) is inoculated by serial dilutions of sample and incubated at 35 $\pm$ 0.5°C for 24 $\pm$ 2 hrs, the tubes were checked for gas production. If the test was positive a "confirmed" test was performed. If the test was negative, the tubes were incubated for another 24 $\pm$ 3 hrs. The tubes that still tested negative after 48 hrs were discarded.

The second phase of this procedure is the confirmed test. The confirmed test is required for water quality determinations since the presumptive test is subject to false positives (USEPA 1978). The medium used in the confirmed test is a brilliant green lactose bile (BGB) which is selective for the coliform group. An inoculating loop is used to transfer from the positive LST tube to the BGB tube. The BGB tube is incubated at 35 $\pm$ 0.5°C and gas production is read at 24 $\pm$ 2 hrs. Again, negative tubes are incubated for another 24 $\pm$ 3 hrs.

At the end of the confirmed test, a count for total coliforms can be obtained. Calculations of the MPN per 100 ml and 95 percent confidence intervals can be taken from tables on pages 924-925 of Standard Methods (APHA 1975). The middle dilution of the three chosen is used to calculate the MPN index. The following is an example:

Dilution	10^0	10^{-1}	10^{-2}	10^{-3}	10^{-4}
Positive tubes	5	5	4	2	0
Code	5-4-2				
MPN/100 ml	$130 * 10^2 = 13000/100$ ml				

Therefore, this sample had 13,000 total coliform per 100 ml of water.

The enumeration of fecal coliforms follows the same procedure as that of the confirmed test except different media and temperatures are used. The EC medium rather than BGB and a temperature of 44.5 $\pm$ 0.2°C rather than 35 $\pm$ 0.5°C are used. The EC tubes are inoculated from positive presumptive tubes (LST) and incubated for 24 $\pm$ 2 hrs. Then they are read and discarded. The same procedure is followed for calculating the MPN index. The third step of the MPN method is the completed test which was not utilized because of time and equipment limitations. Also, it is not required for analysis of nonpotable water.

RESULTS

Model Development

Overland Flow

The kinematic wave approximation was used to model overland flow because it is physically based and yet relatively simple. Though this project used a single plane, applications of the kinematic cascade by Kibler and Woolhiser (1970) and Rovey et al. (1977) have demonstrated that watershed response can be described by a series of planes with uniform shape and roughness.

The derivation of the kinematic wave equations (Eagleson 1970, Overton and Meadows 1976, and Rovey et al. 1977) is normally based on one-dimensional fluid flow. The continuity equation is written

$$\frac{\partial h}{\partial t} + \frac{\partial q}{\partial x} = q_\ell \quad . \; . \; . \; . \; . \quad (3)$$

where

h	=	depth of flow (L)
q	=	flow per unit width ($L^3/T*L$)
q_ℓ	=	lateral inflow (L/T)
x,t	=	space-time coordinates

The energy equation is written

$$\frac{\partial u}{\partial t} + u\frac{\partial u}{\partial x} + g\frac{\partial h}{\partial x} = g(s_o - s_f) - \frac{q_u}{h} \quad . \; . \; . \; . \; . \quad (4)$$

where

s_o	=	slope of runoff surface
s_f	=	friction slope
u	=	velocity (L/T)
g	=	acceleration of gravity (L/T^2)

This set of hyperbolic partial differential equations is known as the Saint Venant equations. In flood routing, they are termed the dynamic equations. For certain conditions (Lighthill and Whitman 1955, Woolhiser and Ligget 1967), the gravity and friction components dominate the unsteady flow terms in Equation 4, and the equation reduces to

$$s_o = s_f \quad . \; . \; . \; . \; . \; . \; . \quad (5)$$

Equation 5, the kinematic wave approximation to the energy equation, allows use of a normal flow equation such as Darcy-Weisbach for laminar flow or Chezy's or Manning's equations for turbulent flow.

In general, these relationships can be expressed as

$$q = ah^m \quad . \; . \; . \; . \; . \; . \; . \quad (6)$$

where

a, m = constants

The coefficient, a, is known as the slope-roughness factor, and m is the channel shape fator. Since overland flow is assumed to occur in a wide rectangular channel, the shape factor, m, is constant depending on the normal flow relationship used. The value of m for Chezy's equation is 1.5; for Manning's equation 1.667; and for Darcy-Weisbach 3.0. The value of the slope-roughness factor can be obtained from the following relationships. For Chezy's equation:

$$a = C\sqrt{s_o} \quad . \; . \; . \; . \; . \; . \; . \; . \quad (7)$$

where

C = Chezy's C

for Manning's equation:

$$a = \frac{1.49 \sqrt{s_o}}{n} \quad . \quad . \quad . \quad . \quad (8)$$

where

n = Manning's n

for the Darcy-Weisbach equation:

$$a = \frac{g s_o}{3v} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (9)$$

where

v = kinematic viscosity

The flow generally goes from laminar to turbulent as one goes down slope. Overton and Meadows (1976) compared results with three friction laws, Chezy, Manning, and Darcy-Weisbach, on 214 dimensionless hydrographs and found prediction errors of 15 percent for Chezy and Manning and 19 percent for Darcy-Weisbach. Lane et al. (1975) utilized a transitional Reynolds number and both flow regimes to fit their hydrographs. For this study, turbulent flow as assumed, and Chezy's equation was used.

For overland flow, the lateral inflow is equal to the rainfall excess; therefore, Equations 3 and 6 have two unknowns: h and q. Substituting Equation 6 into 3 and rewriting, we have

$$\frac{\partial h}{\partial t} + amh^{m-1} \frac{\partial h}{\partial x} = p \quad . \quad . \quad . \quad (10)$$

where

p = rainfall excess (L/T)

For a single overland flow plane, the initial and boundary conditions are

$$h(o,x) = 0 \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (11)$$

$$h(t,0) = 0 \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (12)$$

Since Equation 10 is quasi-linear, an analytical solution can be obtained by the method of characteristics as described by Henderson (1966). The unknowns in Equation 10 are $\partial h/\partial t$ and $\partial h/\partial x$, and solution requires another equation. This equation is obtained by writing the total differential for h

$$dh = \frac{\partial h}{\partial t} dt + \frac{\partial h}{\partial x} dx \quad . \quad . \quad . \quad (13)$$

Combining Equations 10 and 13 in matrix form

$$\begin{bmatrix} 1 & amh^{m-1} \\ dt & dx \end{bmatrix} \begin{bmatrix} \frac{\partial h}{\partial t} \\ \frac{\partial h}{\partial x} \end{bmatrix} = \begin{bmatrix} p \\ dh \end{bmatrix} \quad . \quad (14)$$

Setting the determinant of the coefficient matrix to zero defines the path of the characteristic in the x,t plane

$$\frac{dx}{dt} = amh^{m-1} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (15)$$

This equation defines the velocity of the wave as opposed to the velocity of the water, u.

Given an x,t plane (Figure 29), the characteristic that originates from the point (0,0), for the case of constant input, is termed the limiting characteristic. Below this characteristic, flow is unsteady and uniform, and above this characteristic, flow is steady and nonuniform. The position of the limiting characteristic can be found by solving Equation 15

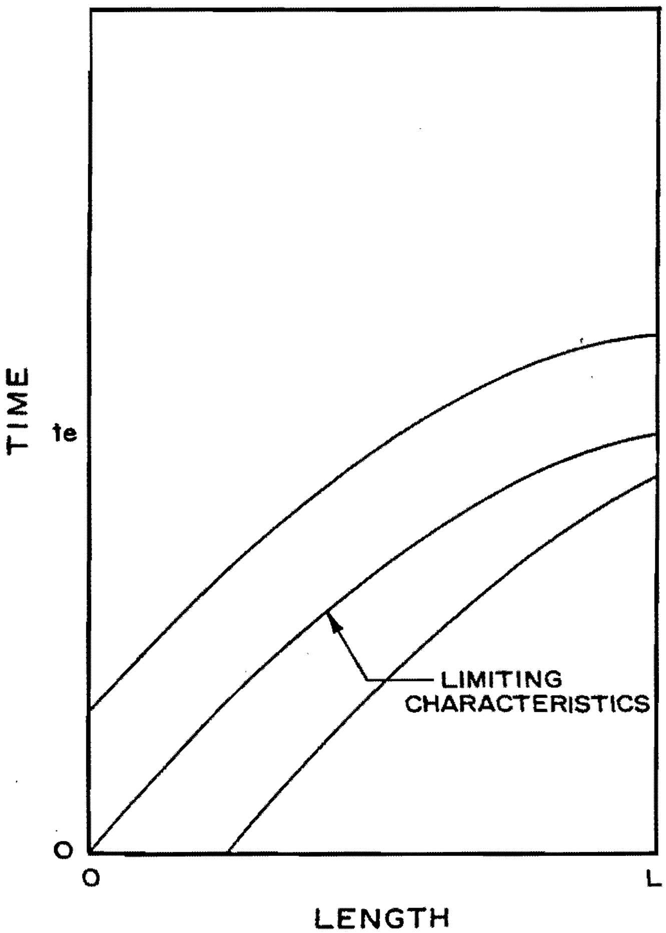

Figure 29. Solution domain for method of characteristics for constant input.

$$x = ap^{m-1}t^m + x_o \quad . \quad . \quad . \quad . \quad . \quad (16)$$

where

x = position of limiting characteristic
x_o = position at $t = 0$

For overland flow, x_o is equal to 0, and Equation 16 becomes

$$x = ap^{m-1}t^m \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (17)$$

By solving for t, the time to steady state or equilibrium at any point x on the plane is found to be

$$t_e = \left[\frac{x}{ap^{m-1}}\right]^{1/m} \quad . \quad . \quad . \quad . \quad . \quad (18)$$

where

t_e = time to equilibrium or steady state

When the flow duration exceeds t_e, both depth and flow remain constant at x as long as the input is constant.

Using Cramer's rule and substituting the right hand side of Equation 14 into the second column of the coefficient matrix, the depth as a function of time is

$$h = h_o + p(t-t_o) \quad . \quad . \quad . \quad . \quad . \quad (19)$$

where

h_o = depth at t_o

For time greater than t_e, Equation 19 becomes

$$h = h_o + p(t_e - t_o) \quad . \quad . \quad . \quad (20)$$

The flow rate is determined by substituting into Equation 6

$$q = a[h_o + p(t - t_o)]^m \quad t \leq t_e \quad (21a)$$

$$q = a[h_o + p(t_e - t_o)]^m \quad t \geq t_e \quad (21b)$$

For the initial and boundary conditions of Equations 10 and 11, h_o and t_o are equal to zero in equations derived for constant rainfall excess. Other cases would require a different characteristic solution or, for more general cases, a numerical solution.

Parameter Estimation for Overland Flow

The conceptual basis of the kinematic wave equation allows the parameter values to be estimated in advance. Once the value of Chezy's C is established, the model should reproduce actual events. The problem is that these roughness factors cannot be determined a priori for most surfaces. Generally, the channel shape factor, m, is held constant and optimization is performed on the slope-roughness factor. Foster (1971) discussed three methods in which the friction may be obtained from observed hydrographs. For this study, the roughness was obtained by matching the observed with the characteristic solution. The shape parameter was held constant at 1.5, hence the Chezy law was utilized.

Pollutant Model

As derived in the Appendix, the continuity equation for pollutant movement in a rectangular channel is

$$\frac{\partial c}{\partial t} + u\,\frac{\partial c}{\partial x} = \frac{-q_\ell}{h}\,c + \frac{b}{h} - \frac{r}{h} \quad . \quad (22)$$

where

c	=	pollutant concentration (M/L^3)
u	=	flow velocity (L/T^2)
q_ℓ	=	lateral inflow (L/T)
b	=	source (M/L^2*T)
r	=	sink term (M/L^2*T)
x,t	=	space-time coordinates
h	=	depth (L)

The pollutant is assumed to move by advection only; dispersion was not considered important (see Equation 2). Also, the velocity of the pollutant is the same as the velocity of the water, not the velocity of the wave as given by Equation 15.

Equation 22 is a linear, hyperbolic, partial differential equation like the Saint Venant equations. Analytical solution by the method of characteristics is possible. The total differential of c is

$$dc = \frac{\partial c}{\partial t}\,dt + \frac{\partial c}{\partial x}\,dx \quad . \; . \; . \quad (23)$$

and writing Equations 22 and 23 in matrix form gives

$$\begin{bmatrix} 1 & u \\ dt & dx \end{bmatrix} \begin{bmatrix} \dfrac{\partial c}{\partial t} \\ \dfrac{\partial c}{\partial x} \end{bmatrix} = \begin{bmatrix} \dfrac{-q_\ell c}{h} + \dfrac{b-r}{h} \\ dc \end{bmatrix} \quad . \; . \; . \; . \quad (24)$$

Setting the determinant of the coefficient matrix equal to zero, the characteristic path is defined by

$$\frac{dx}{dt} = u \quad . \; . \; . \; . \; . \; . \; . \; . \quad (25)$$

As with the overland flow equations, substitution of the right hand side of Equation 24 gives the change in pollutant concentration in time or space, depending on which is desired.

Random Pollutant Model

The above deterministic representations may be adequate in some cases. Often, it is desirable to use random components to account for measurement errors, uncertainty in model components, or a lack of understanding of the system. For coliform transport in overland flow, all three of these factors are prevalent. The MPN method used to enumerate the bacteria lacks precision. Though the velocity of flow is treated deterministically in this analysis, on a soil surface the roughness will vary in essentially a random fashion causing the velocity to vary.

For the present effort, the randomness was introduced into the source-sink terms in Equation 22 by means of Gaussian white noise. Gaussian white noise terms provide randomness in time and space in a framework that permits a mathematical solution. Also, Gaussian white noise approximates the behavior of a number of random processes in nature (Soong 1973).

The white noise terms were assumed to be independent (Finney et al. 1979). Though it was not considered absolutely necessary, this assumption greatly simplified solution. Equation 22, reformulated to include the random components, is

$$\frac{\partial c}{\partial t} + u\frac{\partial c}{\partial x} = \frac{-q_\ell}{h}c + \frac{(b-r)}{h} + \tau + uX \quad . \; . \; . \; . \; . \; (26)$$

where

τ	=	Gaussian white noise term in time with $E[\tau] = 0$ $E[\tau(t)\tau(s)] = \delta(t-s)$ T/t
X	=	Gaussian white noise term in space with $E[X] = 0$ $E[X(y)X(z)] = \delta(y-z)$ $S/(L-X_o)$
δ	=	Dirac delta function
t,s	=	variance terms

Solution of the Random Differential Equation

Solution of the above random differential equation can be in the form of a probability density function (pdf) for the stochastic process, the moments of the stochastic process, the stochastic process itself, or any combination of the three. The advantage in solving for the PDF is that probability statements can be made about the limits being exceeded. Solution for the pdf may not be possible, but the moments of the stochastic process can always be obtained. The mean and variance provide information about the central tendency and the uncertainty in the process respectively.

By rearranging Equation 26, the pdf can be obtained. Recall from Equation 25, that the velocity defines the characteristic in the space-time plane. Solution of Equation 25 for x, assuming u is constant, is

$$x = ut + k(x,t) \quad . \; . \; . \; . \; . \; (27)$$

where

k = constant of integration

In this case, the distance x is a function of t or x(t). The constant k determines where on the X axis the characteristic originates. By solving along a characteristic, we obtain

$$c(t) = c(t,x(t)). \quad . \; . \; . \; . \; . \; (28)$$

The definition for dc/dt then becomes

$$\frac{dc}{dt} = \frac{\partial c}{\partial t} + \frac{\partial c}{\partial x}\frac{dx}{dt} = \frac{-q_\ell}{h}c + \frac{b-r}{h} + \; + uX \quad . \; . \; . \; . \; . \; (29)$$

The original partial differential equation has thus been reduced to an ordinary differential equation in time by solving along a characteristic. The constant term in Equation 27 defines the starting point of the characteristic at t=0, therefore

$$c(t,x(t)) = c(t;k) \quad . \; . \; . \; (30)$$

and

$$\frac{dc(t;k)}{dt} = -\frac{q_\ell}{h}c(t;k) + \frac{b-r}{h} + \tau + uX \quad . \; . \; . \; . \; . \; . \; (31)$$

This solution process can easily be reformulated to determine concentration with distance. There were two reasons for using time rather than distance for this analysis. First, steady flow conditions would have had to have been assumed. This has been done in sediment modeling (Foster et al. 1980), but pollution loading concentrates on the rising hydrograph where it is important to consider unsteady conditions. Second, the only point on the slope of the experimental surface at which measurements were taken was at the slope outfall. Thus, it would be difficult to verify the model at points along the slope. Measurements were made over time at the slope outfall, and the material washed from the slope determines the environmental impact.

Solution of this random ordinary differential equation is presented by Soong (1973). The Fokker-Planck equation describes the transitional pdf

$$\frac{\partial f}{\partial t} = \frac{\partial}{\partial c}\left[\left(\frac{q_\ell}{h}c + \frac{(b-r)}{h}\right)f\right] + \frac{1}{2}\left(\frac{T}{t} + \frac{uS}{(L-X_o)}\right)\frac{\partial^2 f}{\partial_c{}^2} \quad . \quad (32)$$

where

f	=	$(f(c,t\mid b;k))$ = transitional probability density function
t	=	time (T)
L	=	slope length (L)
X_o	=	distance from top of slope to location of pollutant (L)

Solution of this equation requires the following initial and boundary conditions

$$f^*(c,o;k) = f^*(b;k) \quad . \; . \; . \; (33a)$$

$$_{-\infty}\int^{\infty} f(c,t;k) = 1 \quad . \; . \; . \; (33b)$$

Equation 33a is the initial condition and b is introduced as a random variable describing the initial condition. Since the equation defines the transitional pdf, the initial condition can be rewritten as

$$f(c,o\mid b;k) = f^*(b,o;k) \quad . \; . \; (34)$$

Within the given initial and boundary conditions, the solution for the pdf from Equation 32 becomes

$$f(c,t;k) = {}_{-\infty}\int^{\infty} f(c,t\mid b;k)f^*(b;k)db \quad . \; . \; . \; (35)$$

From Soong (1973), the transitional pdf is determined as

$$f(c,t\mid b;k) \sim N(\bar{c}(T,b;k),G(t;k)) \quad . \; . \; . \; . \; (36)$$

The transitional pdf is a normal distribution.

Integration of Equation 35 requires the initial distribution and estimates of the mean and variance terms for the transitional pdf. By taking expected values of Equation 31, equations for the mean, $\bar{c}(t,b;k)$, and variance, $G(t;k)$ can be obtained.

$$\frac{d\bar{c}(t;k)}{dt} = \frac{-q_\ell}{h}\bar{c}(t;k) + \frac{(b-r)}{h} \quad (37)$$

where

$\bar{c}(t;k)$ = mean at time t

$$\frac{dG(t;k)}{dt} = \frac{-2q_\ell}{h}G(t;k) + \frac{T}{t} + \frac{uS}{(L-X_o)} \quad . \; . \; . \; (38)$$

where

$G(t;k)$	=	variance at time t
T,X	=	variance terms defined by white noise

Solution of these equations for the case of overland flow with constant rainfall excess leads to three cases. To determine which case is appropriate, it is necessary to define a travel time, t^*, from the kinematic wave equation. The assumption made in the derivation of Equation 22 was that the pollutant had the same velocity as the water. For the case of constant rainfall excess, the velocity is written as

$$u = \frac{q}{h} = \frac{ap^m t^m}{pt} = ap^{m-1}t^{m-1} \qquad t \leq t_e \quad . \; . \; . \; (39a)$$

$$u = \frac{q_e}{h_e} = \frac{ap^m t_e^{\,m}}{p\, t_e} = ap^{m-1} t_e^{\,m-1}$$

$$t \geq t_e \quad . \quad . \, (39b)$$

where

u	=	velocity (L/T)
q	=	discharge per unit width ($L^3/T*L$)
h	=	depth (L)
p	=	rainfall excess (L/T)
t	=	time (T)
a,m	=	kinematic wave parameters

A pollutant located a distance, X_o, from the top of the slope will require a time to travel to the slope outlet that can be estimated as

$$t^* = \left[\frac{m(L-X_o)}{ap^{m-1}}\right]^{1/m} \qquad X_o \geq X_L$$

$$. \quad . \quad . \quad . \, (40a)$$

$$t^* = ts + \frac{m(L^{1/m} - X(ts)^{1/m})}{(ap^{m-1})^{1/m}}$$

$$X_o \leq X_L \quad . \quad . \, (40b)$$

where

ts	=	time that the pollutant characteristic intercepts the limiting characteristic
X(ts)	=	distance from X_o to the intersection of the limiting and particle characteristics
L	=	slope length (L)

The value for ts is defined by equating Equations 16 and 39a and solving

$$ts = \left[\frac{mX_o}{a(m-1)p^{m-1}}\right]^{1/m} \quad . \quad . \, (40c)$$

The distance, above which $t^*>te$, was found by equating Equations 18 and 40a

$$X_L = \frac{L(m-1)}{m} \quad . \quad . \quad . \, (41)$$

where

m	=	constant from friction law assumption

For a value of X_o less that that given by this equation, the travel time (t^*) is greater than equilibrium time (t_e).

These equations permit definition of three different cases. The definitions are based on the relationships between t^*, t_e, and time of cessation of rainfall (t_r), and the three cases are:

Case 1: $t^* \leq t \leq t_e$ or $t^* \leq t \leq tr$
Case 2: $t_e \leq t \leq tr$
Case 3: $t_e \leq t^* \leq t \leq tr$

Case 1 represents the situation after the pollution has reached the outlet but while the flow is still increasing. Case 2 extends Case 1 for the period after equilibrium flow conditions are achieved. Case 3 occurs when the pollution doesn't reach the outlet until after equilibrium is achieved.

For solution of Equations 37 and 38 for each of these cases, an initial condition is needed. For Cases 1 and 3, the initial condition at the slope outlet is the same, namely:

$$\bar{c}\ (t^*;k) = 0 \quad . \quad . \quad . \quad . \quad . \, (42)$$

$$G\ (t^*;k) = 0 \quad . \quad . \quad . \quad . \quad . \, (43)$$

Prior to the arrival of the pollutant no concentration is observed. The initial condition for Case 2 will be discussed with the solution of that case.

<u>Case 1</u>

This is the zone of flow establishment (Hjelmfelt 1976) in which the flow is unsteady and uniform. The velocity is described by Equation 39a. The solution for the mean using Equation 42 and integrating between t^* and t_e or t^* and t_r, whichever is pertinent, is

$$\bar{c}(t;k) = \frac{(b-r)}{p} \frac{(t-t^*)}{t}$$

$$t^* \leq t \leq t_e$$

$$t^* \leq t \leq t_r \quad . \quad . \quad . \quad (44)$$

The solution for the variance is found from Equation 43 and the same limits of integration

$$G(t;k) = \frac{T}{2}\left[\frac{t^2-t^{*2}}{t^2}\right] + \frac{ap^{m-1}S}{(L-X_o)(m+2)}\left[\frac{t^{m+2}-t^{*m+2}}{t^2}\right]$$

$$t^* \leq t \leq t_e$$

$$t^* \leq t \leq t_r \quad . \quad . \quad . \quad (45)$$

<u>Case 2</u>

Hjelmfelt termed this the zone of established flow in which discharge is steady but nonuniform. A different set of initial conditions is required in order to maintain continuity. If Equations 44 and 45 are solved at equilibrium time (t_e), the initial conditions are

$$\bar{c}(t_e;k) = \bar{c}(t_e) \quad . \quad . \quad . \quad . \quad . \quad (46)$$

$$G(t_e;k) = G(t_e) \quad . \quad . \quad . \quad . \quad . \quad (47)$$

Solving Equations 42 and 43 using t_e to t_r as the limits of integration, the solutions for the mean and variance become

$$\bar{c}(t;k) = \frac{(b-r)}{p}\left[1-\exp\left[1-\frac{t}{t_e}\right]\right] + \bar{c}(t_e)\exp\left[1-\frac{t}{t_e}\right]$$

$$t_e \leq t \leq t_r \quad . \quad . \quad . \quad (48)$$

$$G(t;k) = \left(\frac{T}{2} + \frac{ap^{m-1}St_e^{\ m}}{(L-X_o)^2}\right)\left[1-\exp\left(\frac{2}{t_e}(t_e-t)\right)\right] + G(t_e)\left[\exp\left(\frac{2}{t_e}(t_e-t)\right)\right]$$

$$t_e \leq t \leq t_r \quad . \quad (49)$$

<u>Case 3</u>

This final case is represented by the fact that equilibrium exists prior to the arrival of the pollutant at the outlet. This situation is defined by Equation 41 when X_o is less than the distance indicated by solution of the equation for the friction law assumed. The initial conditions are the same as those for Case 1 (Equations 42 and 43), but the limits of integration are t^* to t_r. What is different is that equilibrium conditions exist for depth and velocity. The solution for the mean is

$$\bar{c}(t;k) = \frac{(b-r)}{p}\left[1-\exp\left[\frac{t^*-t}{t_e}\right]\right]$$

$$t_e \leq t^* \leq t \leq t_r$$

$$. \quad . \quad . \quad (50)$$

The variance is defined by

$$G(t;k) = \left(\frac{T}{2} + \frac{ap^{m-1}St_e^{\ m}}{2(L-X_o)}\right)$$

$$\left[1 - \exp\left(\frac{2}{t_e}\ (t^*-t)\right)\right]$$

$$t_e \leq t^* \leq t \leq t_r \quad . \quad . \quad (51)$$

Of the above cases, Case 1 ($t^* \leq t_e$) is the most important since natural rainfall events are seldom of sufficient duration or constant intensity to allow overland flow to reach equilibrium. For feedlot situations, Cases 2 and/or 3 could be important and warrant consideration.

Solution for the Distribution

The solutions for the mean and variance make possible the solution for the joint probability density function for time and coliform at the slope outlet. In order to integrate Equation 35, an initial distribution is required. Two different initial distributions were assumed. First, the initial distribution was assumed to be constant, and in the other case, the distribution was assumed to be normal.

The constant initial distribution, defined by the Dirac delta function, is for Equation 34

$$f(c,0|b;k) = \delta(c-b) \quad . \quad . \quad . \quad (52)$$

Substituting Equation 52 into Equation 35, the resulting distribution after integration is normal (Soong 1973)

$$f(c,t;k) \sim N(\bar{c}(t;k),\ G(t;k)) \quad . \quad (53)$$

where

$\bar{c}(t;k)$ = mean defined by Equation 37

$G(t;k)$ = variance defined by Equation 38

The probability of a MPN value occurring at time t can be found by integrating Equation 53 or

$$\int_{-\infty}^{S} \frac{1}{(2\pi G(t;k))^{1/2}}\ \exp\left[\frac{-1}{2G(t;k)}\right.$$

$$\left.(c-\bar{c}(t;k))^2\right] dc \quad . \quad . \quad . \quad (54)$$

Since the distribution is normal, tabulated values are available.

The resulting joint pdf for the case of the normal initial distribution is derived in the Appendix for Case 1. The solutions for Cases 2 and 3 are similar since Equations 37 and 38 are linear. The resulting joint pdf is normal

$$f(c,t;k) = \frac{1}{(2\pi(H^2V+G))^{1/2}}\quad \exp$$

$$\left[\frac{-1}{2}\ \frac{(c-(\bar{b}-r)H)^2}{H^2V+G}\right]$$

$$t^* \leq t \leq t_e \quad . \quad . \quad . \quad (55)$$

where

H	=	$[t-t^*/pt]$
$\bar{b}$	=	mean of normal initial distribution
r	=	bacterial loss parameter
G	=	variance of process at time t
V	=	variance of normal initial distribution

For either initial distribution, the resulting joint pdf is normal, which makes determination of probabilities much easier.

Summary of Model Development

The previous sections describe the development of a stochastic mathematical model to predict the overland movement of fecal indicator bacteria from source material to channel systems. Time and space randomness were introduced through two independent Gaussian white noise terms. Since the result was a linear, constant coefficient, ordinary differential equation and independent Gaussian white noise terms were used, the resulting pdf at the slope outlet had a normal distribution regardless of whether a normal distribution or a constant was assumed to be the initial condition. With the pdf at the slope outlet being normal, the mean and variance completely specify the distribution. Estimation of the means and variances depended on the relationship between the travel time from the bacteria source to slope outlet and the equilibrium time of the overland flow.

Figure 30 schematically represents the model. Because of the form of the governing differential equation, solution for the pdf was possible. In other circumstances, solution for the pdf may not be possible, that box would have to be removed, and only the mean and variance would be available for a management decision. Whether the full pdf or just the mean and variance were made available, the management decisions would be based on information on the uncertainty in the system and not on a single deterministic number.

Data Analysis

Hydraulic Variables

In order to have a common time basis, the times noted for runoff from the plot had to be corrected for travel time over the triangular concrete transition and through the HS flume.

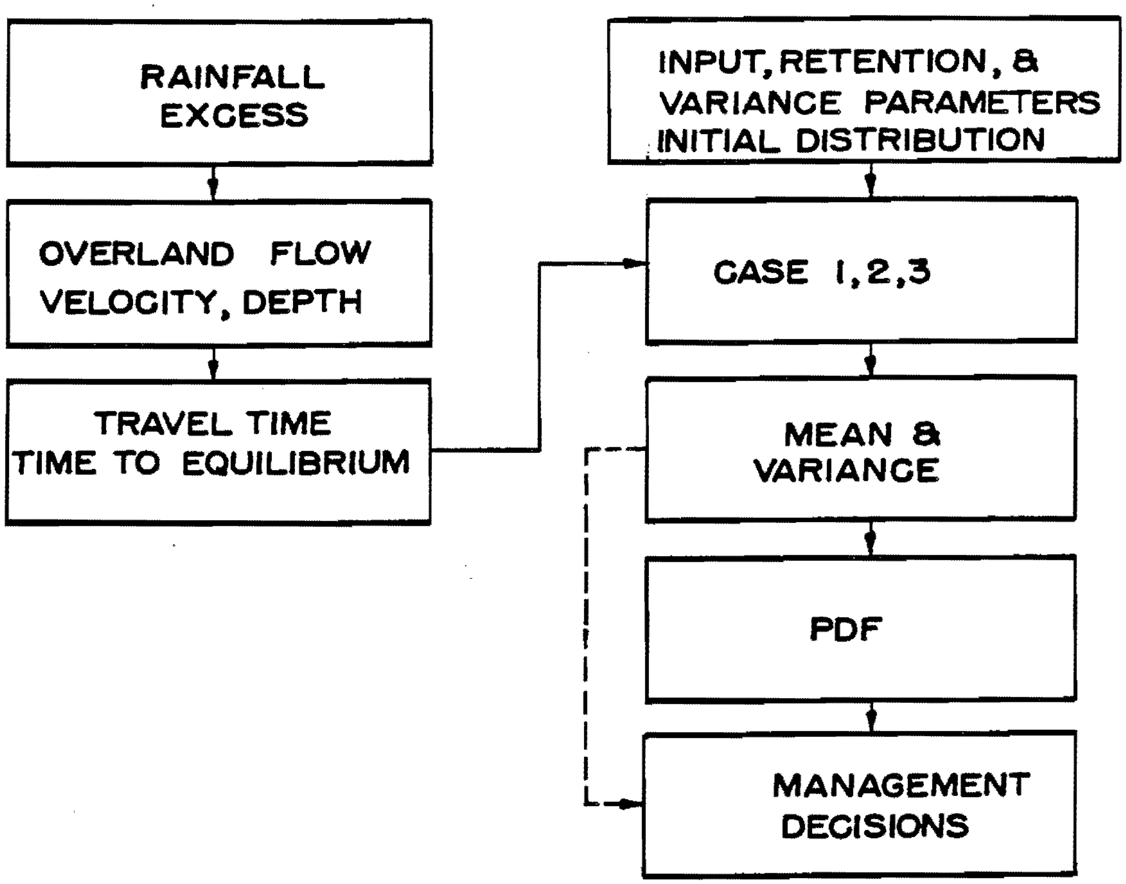

Figure 30. Schematic representation of random coliform transport model.

The procedure used a simple storage routing (Wu et al. 1978) to correct for travel time down the flume, and the kinematic wave equations for a converging surface were used for the transition. The problem with using the kinematic wave equations was in estimating the roughness. Therefore, trials were conducted using various values of Chezy's C for smooth concrete surfaces and different rainfall intensities observed during simulation. It was found that the travel time through the transition varied from 7 sec at the start of the run down to less than 3 sec at the end. These durations are well within the experimental errors associated with the sampling duration (the period over which a sample is collected) and with clock reading. Hence, travel time through the transition was ignored. In contrast, the storage by the flume was significant, particularly in the early portion of the hydrograph, and these corrections were applied. The observed and corrected flow are listed in the Appendix.

Optimization of the slope-roughness parameter in Equation 6 was done by matching the observed hydrograph with the characteristic solution and minimizing the sum of squares. The results are presented in Table 7. As a matter of comparison, Rovey et al. (1977) presented a range of Chezy C values for a concrete surface of 403.0-209.8 $cm^{1/2}/sec$. Using a slope of 0.06 for this plot, the range in Chezy C for this simulation is 342.5-92.3 $cm^{1/2}/sec$. The effects of rainfall intensity were not incorporated into this analysis.

Coliform Model

For each case, three coefficients were required to estimate the mean and the variance. The travel time (t^*) and time to equilibrium (t_e) were calculated from the mean hydraulic variables for each source distance.

For the present effort, all coefficients were assumed to be constants since there is no information available on their time variation. The most obvious process for which there is a total lack of information is the movement of the bacteria from the source to the flow or the input parameter b. Khaleel et al. (1979) described the erosion of surface applied fecal material with a modified USLE, but even with this, some type of an enrichment ratio would be needed to enumerate the bacteria attached to an eroded particle. Also, this approach would not consider free-flowing bacteria in the water column regardless of how they entered the water column. Of course, it is not known for the overland flow case what percentage of the total bacteria load is transported as free-flowing particles. Studies already described are an attempt to bypass these problems by defining bacterial loads which enter the flow to be transported. The current study considered only fresh material because it was believed to be the most erodible and to be the greatest potential contributor of coliforms.

The input value, b, in Equation 26, was set at $6.0 * 10^6$ MPN/100 ml based on results previously described. Using data from Part I of this study, it would be possible to determine input values for fecal material of different ages and model the response of this material to rainfall because once the bacteria are in the flow, they respond the same regardless of the source. It may be expected that the sample variance from these data may provide the initial variance or the T parameter for Equation 38. The exact interpretation of the variance terms, T and S, in Equation 38 is difficult, but essentially they represent the variance per unit time and space, respectively. Therefore, their integration would be the cumulative noise in the system at that time and point in space.

To complete the modeling effort, the retention coefficient and its variability in time and space, r, T, and

Table 7. Least squares fits of slope roughness parameter for 44 runs.

Date	Source Distance (m)	Rainfall Intensity (cm/min)	a ($cm/sec^{\frac{1}{2}}$)	R^2
6-27	1.52	0.066	43.4	0.85
6-29	1.52	0.073	22.6	0.73
7-01	1.52	0.056	64.8	0.96
7-02	1.52	0.072	29.4	0.33
7-03	3.05	0.078	38.6	0.82
7-04	3.05	0.064	63.6	0.78
7-21	3.05	0.040	54.4	0.79
7-22	3.05	0.043	61.9	0.88
7-26	3.05	0.038	54.6	0.71
7-05	6.10	0.066	58.2	0.83
7-07	6.10	0.066	53.9	0.99
7-10	6.10	0.064	52.3	0.97
7-27	6.10	0.038	63.4	0.73
7-28	6.10	0.040	46.9	0.54
7-29	9.14	0.038	58.9	0.92
7-30	9.14	0.043	62.4	0.81
7-31	9.14	0.041	67.2	0.79
8-01	9.14	0.040	67.7	0.99
8-02	9.14	0.042	62.8	0.76
8-04	12.19	0.038	72.0	0.74
8-05	12.19	0.044	57.0	0.82
8-06	12.19	0.040	63.7	0.92
8-07	12.19	0.038	67.6	0.75
8-08	12.19	0.042	70.3	0.90
7-06	15.24	0.072	48.7	0.97
7-08	15.24	0.066	60.3	0.99
7-15	15.24	0.043	48.9	0.80
8-09	15.24	0.038	58.6	0.52
8-10	15.24	0.043	52.1	0.56
7-11	22.86	0.068	46.8	0.64
7-13	22.86	0.042	56.2	0.99
7-18	22.86	0.038	66.4	0.93
8-11	22.86	0.034	83.9	0.72
8-12	22.86	0.040	56.0	0.85
7-12	27.43	0.048	54.7	0.90
7-14	27.43	0.038	58.9	0.86
7-17	27.43	0.063	48.3	0.95
8-13	27.43	0.038	61.3	0.77
8-14	27.43	0.040	56.2	0.85
7-19	30.48	0.035	83.1	0.73
7-20	30.48	0.038	58.4	0.83
8-15	30.48	0.038	65.0	0.51
8-16	30.48	0.037	67.4	0.80
8-17	30.48	0.037	67.0	0.93

S were estimated. All three parameters were assumed to be constants.

The retention coefficient, r, represents the elimination of coliforms by adsorption to interfaces or clumping and settling of the bacteria. Hendricks et al. (1979) examined the assumption of r being constant in time by measuring the response pattern of Staphylococcus aureus to different adsorbents and found that equilibrium was reached in 20 to 40 min. For an overland flow surface, especially one with a significant clay fraction and depression storage, the number of adsorption sites may be unlimited. Also, only a portion of the total bacterial load may be in contact with the surface at any one time. Therefore, the retention concept is not so limiting an assumption.

Further evidence supporting the assumption can be seen in the calculations using relationships and coefficients from Reddy et al. (1981). A linear form of the Freundlich isotherm describing the number of bacteria adsorbed

$$c_{ad} = K * c_{sol} \quad . \quad . \quad . \quad . \quad . \quad (56)$$

where

c_{ad} = number of adsorbed cells (number/g)

c_{sol} = number of cells in solution (number/ml)

K = retention of coefficient (ml/g)

Reddy et al. (1981) presented values of the retention coefficient for a number of bacteria including a value of 1909.0 ml/g for fecal coliforms in river sediments. The total bacterial population is equal to

$$c_{tot} = cad + csol \quad . \quad . \quad . \quad . \quad (57)$$

and using Equation 56, the total population becomes

$$c_{tot} = (K + 1)\ csol \quad . \quad . \quad . \quad (58)$$

Rearrangement of Equation 58 and substitution of the first order decay relationship for c_{sol} results in

$$c_{sol} = (ctot \exp(-kt))/(K + 1) \quad . \quad . \quad . \quad . \quad . \quad (59)$$

where

c_{tot} = initial population at t=0

k = first order reaction rate (1/day)

t = time in days

K = retention coefficient

If a value for the first order reaction rate is 0.67 (1/day) (Van Donsel et al. 1967) and c_{tot} has a value of $6 * 10^6$ MPN/g effluent, the reduction in counts computed by Equation 59 for a 20 min period can be seen in Table 8. The counts remained essentially constant for the period, indicating that assumption of a constant loss is reasonable.

Estimates of r, T, and S were made by least squares analysis of the solutions for the means and variances of the observed data. All calculations were

Table 8. Results of Equation 59 with first order reaction rate coefficient of 0.67 (1/day), $c_{tot} = 6 * 10^6$ MPN/g effluent, and retention coefficient of 1909.0 ml/g.

Time (min)	Fecal Coliform in Solution (MPN/ml)
5	3134
10	3126
15	3120
20	3112

done using the mean rainfall and hydraulic variables from the replications for each source distance (Table 9), and a minimum of three observations for each case. Use of the mean rainfall and hydraulic variables presented problems in some instances because of changing rainfall intensity over the experimental period (Table 7). If replications for a given distance were run on successive days, the hydraulic and rainfall parameters would be consistent. A period of several days between runs as in the 27.4 m (90 ft) source distance could result in considerable differences between the parameter values. The result in the 27.4 (90 ft) case was that the travel time for one run was considerably shorter than the other four, and the mean fecal coliform counts reflected this. Still, the mean represented the best measure for this effort.

Values for the retention parameter, r, in Equation 37 were determined by using the value for b, the input parameter previously determined as $6.0 * 10^6$ MPN/100 ml and least square analysis of the appropriate solution, Cases 1-3, for the source distances. Results of the analysis are presented in Table 10. The values appear to have a narrow range,

Table 9. Mean rainfall, slope-roughness parameter, travel time, and equilibrium time for concrete plot replications.

Source Distance (m)	p (cm/min)	a	t* (sec)	t_e (sec)
1.52	0.068	40.05	23.52	173.32
3.05	0.053	54.62	43.07	152.58
6.10	0.055	54.94	67.19	149.93
9.14	0.041	63.80	87.92	149.72
12.19	0.040	66.12	104.35	146.68
15.24	0.052	53.72	127.52	154.48
22.86	0.044	61.86	161.56	148.59
27.43	0.045	55.88	201.39	157.84
30.48	0.037	68.18	221.98	147.99

Table 10. Results of least squares fits for coliform removal parameter, r, for concrete runoff surface (values are in (cm * MPN/100 ml * sec)).

Source Distance (m)	Case 1	Case 2	Case 3
1.52	5999990.0	5999998.0	
3.05	5999974.0	5999995.0	
6.10	5999999.0	6000000.0	
9.14	5999998.0	5999999.0	
12.19	a	5999993.0	
15.24	a	5999998.0	
22.86			5999998.0
27.43			a
30.48			a

[a]Insufficient data values (< 3) available for analysis.

and the smooth surface does not have a very high adsorption potential. For modeling, the mean of these values was used, which was 5999993.0 (cm * MPN/100 ml * sec).

Estimates of T and S could not be made for each case. The only case in which estimates of these parameters could be made was Case 1, but this is the most important situation in natural rainfall-runoff systems. To fit values of T and S, a multiple regression was used. Observed variances for 1.52 m - 15.24 m (5 - 50 ft) for Case 1 time intervals were used as dependent variables. There were a total of 15 observations. The resulting equation was

$$s^2 = 128865710.0 - 185947570.0 * X1 - 7733074.0 * X2 \quad . \quad . \quad (60)$$

where

s^2 = observed variances $(MPN/100\ ml)^2$
$X1$ = (t^*/t^2)
$X2$ = $(t^{m+2} - t^{*m+2})/t^2$

The multiple coefficient of determination (R^2) of Equation 60 was 0.12 and statistically, the equation was not significant. Both the intercept or constant term and the coefficient for X1 represent T/2, and they are of the same order of magnitude. Therefore, the constant term was used as the estimate for T/2. The coefficient for X2 was the value used for S.

Model Performance

The MPN Method

Little has been said about interpretation of the MPN index used in this study. It has been noted that the MPN method is statistically based. The index is presented in Standard Methods (APHA 1975) with 95 percent confidence intervals. Since the five-tube, five-dilution method was used, the precision obtained in this study was as great as possible with the method. Still, the confidence intervals should be considered when interpreting the results.

Means and Variances

The observed (Table 11) and predicted means are compared in Figures 31-39. For times prior to travel time (t*), the model assumed predicted coliform counts of zero. For the concrete runoff surface, fecal coliform counts observed prior to t* (Appendix Tables 17-25) were assumed to be

Table 11. Log transformed means of fecal coliform data over time for all source distances (values are ln(MPN/100 ml)).

Source Distance (m)	Time (sec)					
	90	120	150	180	240	300
1.52	5.16	8.76	7.77	8.34	7.81	8.13
3.04	4.70	9.77	9.11	9.14	8.42	9.03
6.10	1.86	7.06	8.50	8.64	7.99	7.82
9.14	1.39	4.21	7.21	7.92	7.86	7.66
12.19	-0.92	2.95	7.95	9.01	8.38	9.03
15.24	0.00	1.34	5.57	7.17	8.99	9.38
22.86	0.79	0.79	2.45	4.23	6.72	7.59
27.43	2.98	2.10	2.58	6.36	8.70	8.59
30.48	1.28	-0.22	0.79	1.61	5.23	6.43

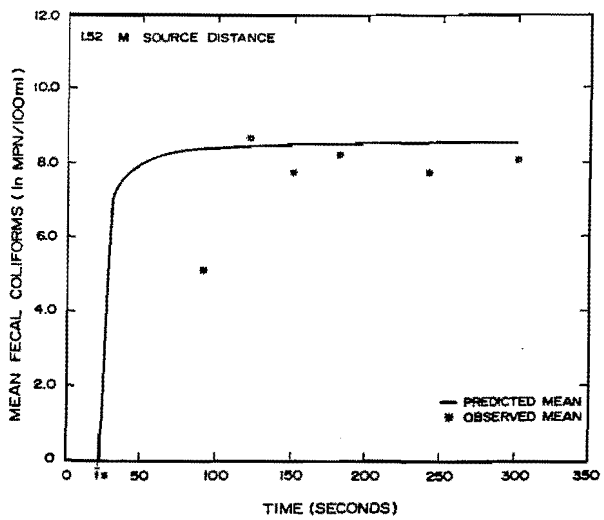

Figure 31. Observed and computed means for 1.52 m (5 ft) source distance.

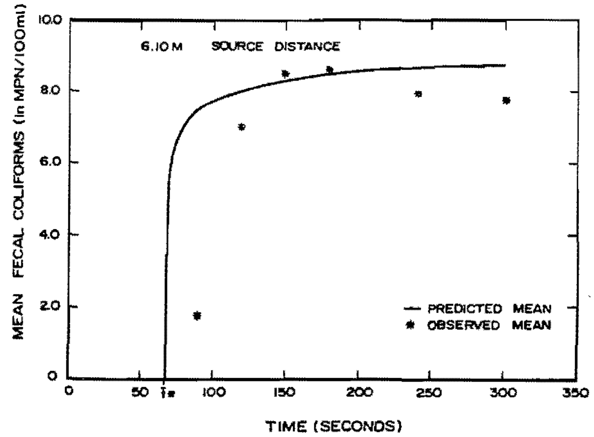

Figure 33. Observed and computed means for 6.10 m (20 ft) source distance.

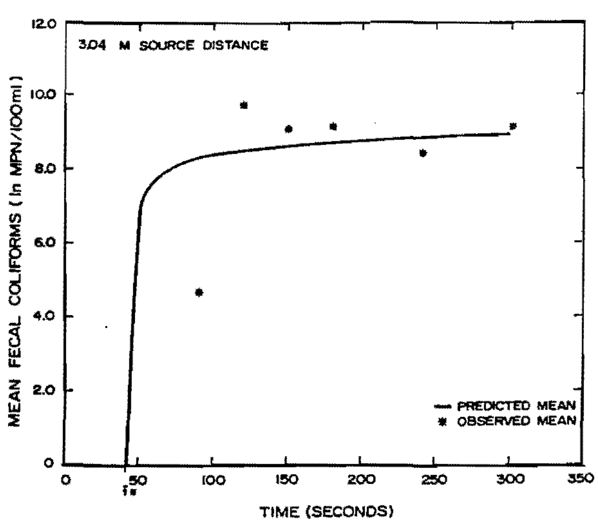

Figure 32. Observed and computed means for 3.04 m (10 ft) source distance.

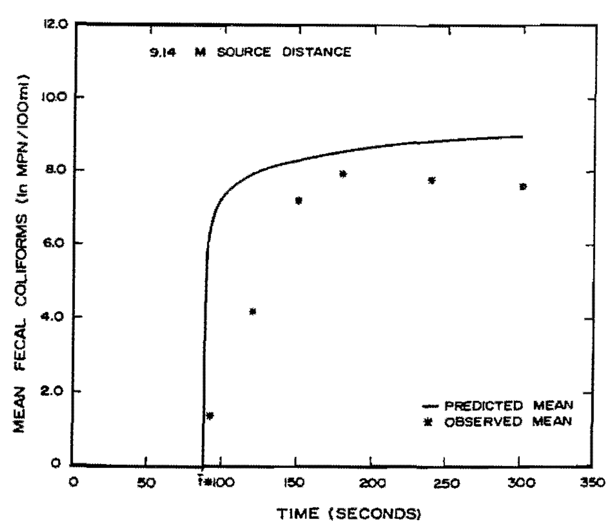

Figure 34. Observed and computed means for 9.14 m (30 ft) source distance.

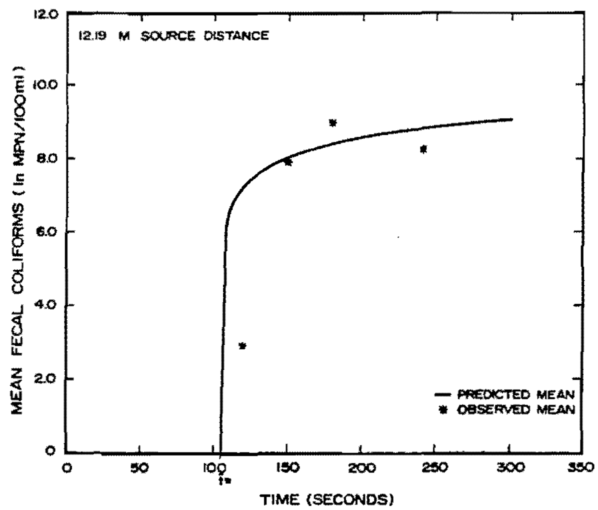

Figure 35. Observed and computed means for 12.19 m (40 ft) source distance.

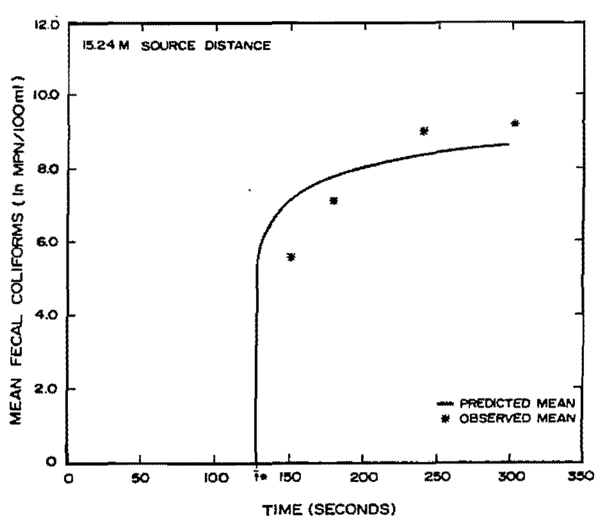

Figure 36. Observed and computed means for 15.24 m (50 ft) source distance.

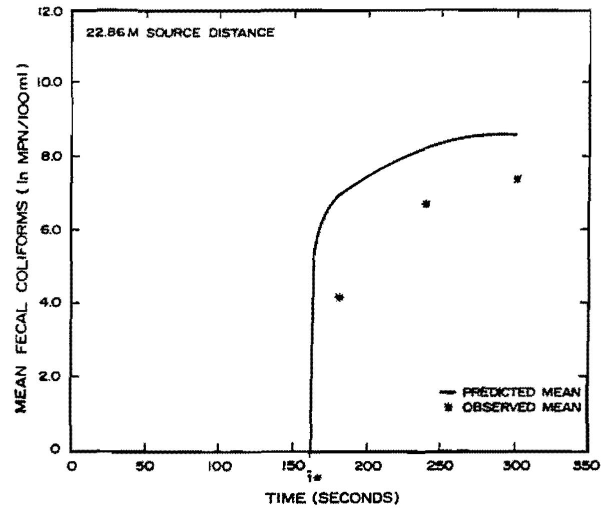

Figure 37. Observed and computed means for 22.86 m (75 ft) source distance.

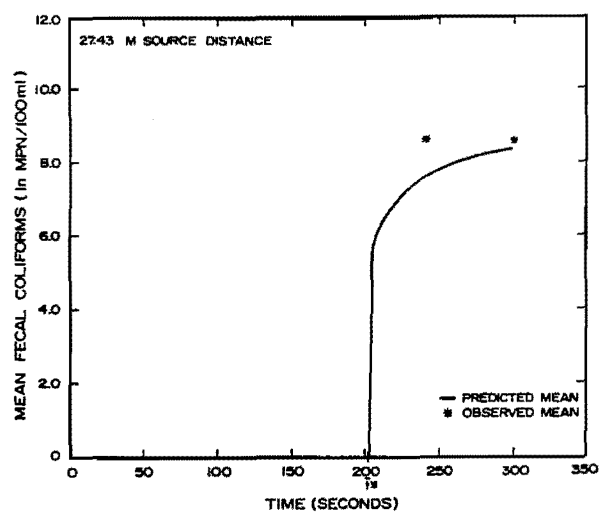

Figure 38. Observed and computed means for 27.43 m (90 ft) source distance.

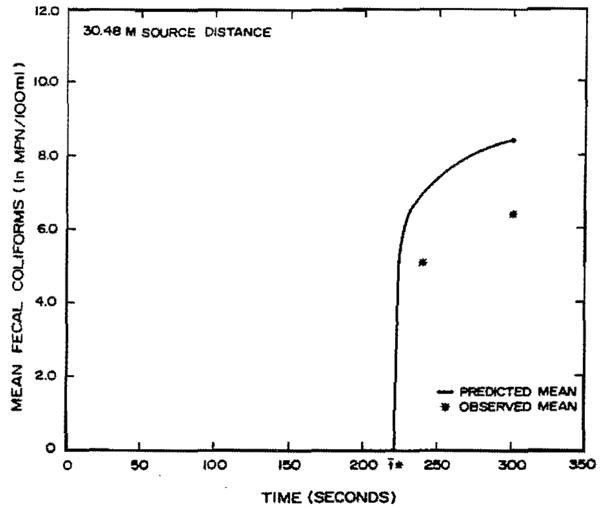

Figure 39. Observed and computed means for 30.48 m (100 ft) source distance.

from inadequate cleaning of the surface or the flume. In most cases, these counts were low (<2 MPN/100 ml). However, on a soil surface, the bacteria may be found at any point on the plane, and the model would have to account for this.

When considering the relationships presented in Figures 31-39, it should be noted that the values were logarithmically transformed for their presentation. The transformation is common for bacterial data.

From Figures 31-39, it is apparent that the model reaches a steady state more rapidly than does the data (Table 11). A steady state was achieved for the closer cases, which indicates that a steady state value could be used for determining the mean of the pdf. For the source distances greater than 15.24 (50 ft), the model simulated the increasing mean counts observed in the data (Figures 36-39).

The observed variances also attained a relatively constant value early in the run (Table 12). Comparison of the observed and predicted values (Figures 40-48) showed that the model predicted a more rapid rise in the variance than was experienced. Again, the steady state value was achieved during the Case 1 time interval indicating that a constant mean and variance could be used to describe the normal distribution.

Equilibrium flow conditions were important in this study and may be important in urban or feedlot hydrologic systems. For natural range systems, natural rainfalls seldom continue constant for a sufficient duration to establish equilibrium conditions. Case 1 conditions were not simulated very well for either the mean or the variance, but the observed data indicated that both the means and variances

Table 12. Log transformed variances for fecal coliform data over time for all source distances (values are $\ln(\mathrm{MPN}/100\ \mathrm{ml})^2$).

Source Distance (m)	Time (sec) 90	120	150	180	240	300
1.52	10.39	18.74	15.30	16.48	14.40	15.29
3.04	8.72	20.04	18.17	17.16	16.54	17.73
6.10	4.16	13.69	16.86	17.05	15.96	16.09
9.14	2.64	8.26	14.18	15.25	13.11	14.74
12.19	-0.23	5.41	16.28	18.23	14.95	17.95
15.24	1.61	0.99	10.37	14.16	18.25	17.70
22.86	1.16	3.19	5.47	8.76	13.34	13.90
27.43	6.58	4.42	6.06	13.75	18.47	18.51
30.48	3.47	0.19	3.19	4.04	9.20	12.86

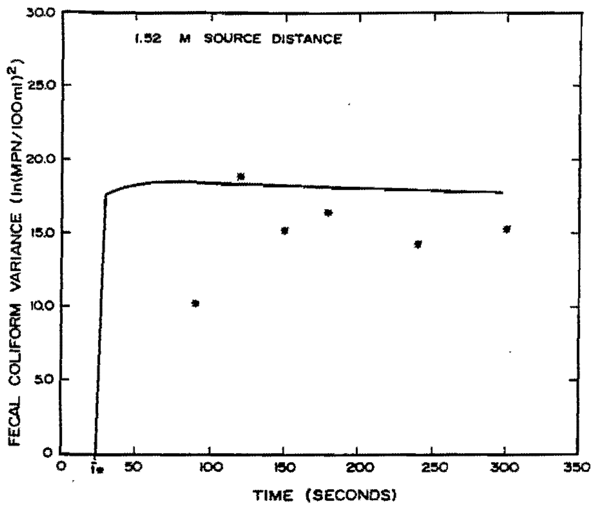

Figure 40. Observed and computed variances for 1.52 m (5 ft) source distance. Variances were computed assuming a Dirac delta initial distribution.

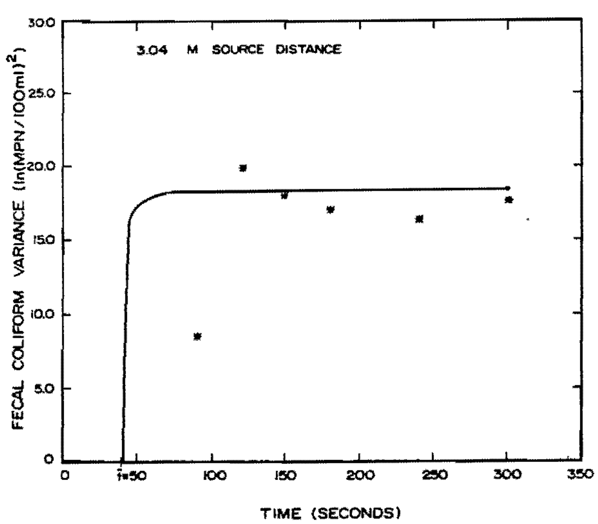

Figure 41. Observed and computed variances for 3.04 m (10 ft) source distance. Variances were computed assuming a Dirac delta initial distribution.

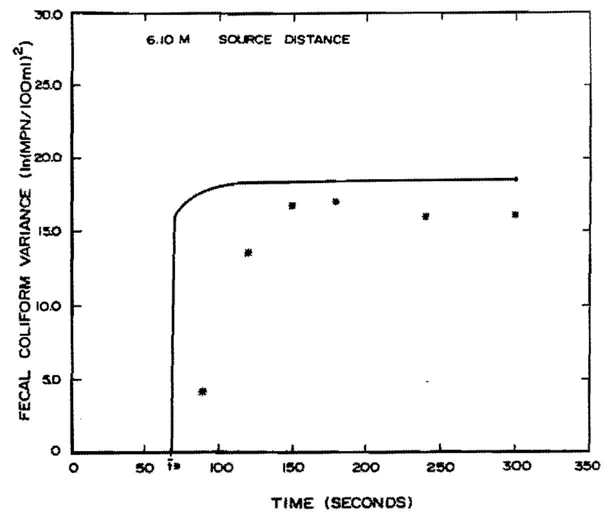

Figure 42. Observed and computed variances for 6.10 m (20 ft) source distance. Variances were computed assuming a Dirac delta initial distribution.

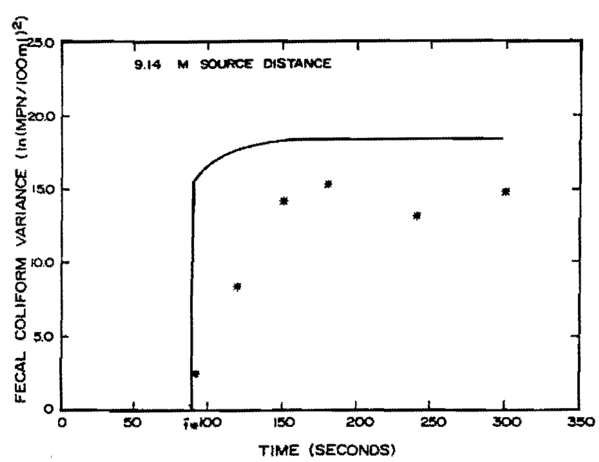

Figure 43. Observed and computed variances for 9.14 m (30 ft) source distance. Variances were computed assuming a Dirac delta initial distribution.

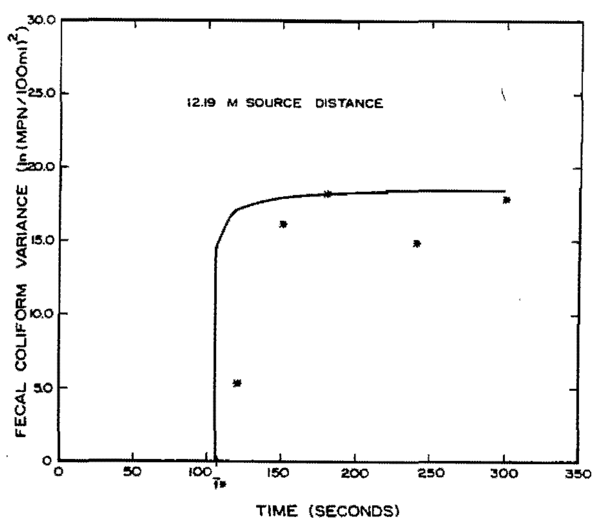

Figure 44. Observed and computed variances for 12.19 m (40 ft) source distance. Variances were computed assuming a Dirac delta initial distribution.

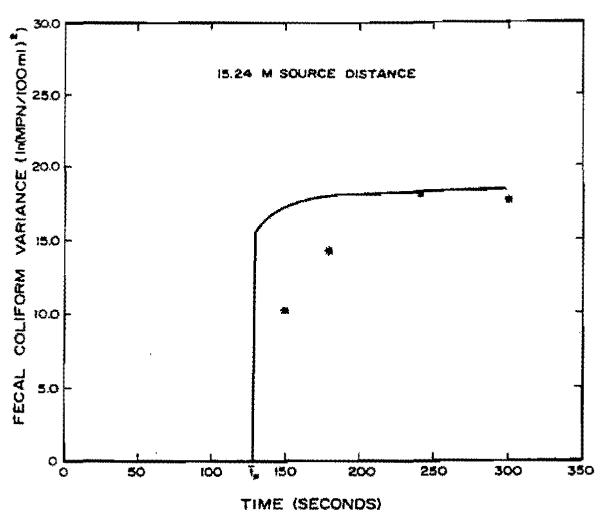

Figure 45. Observed and computed variances for 15.24 m (50 ft) source distance. Variances were computed assuming a Dirac delta initial distribution.

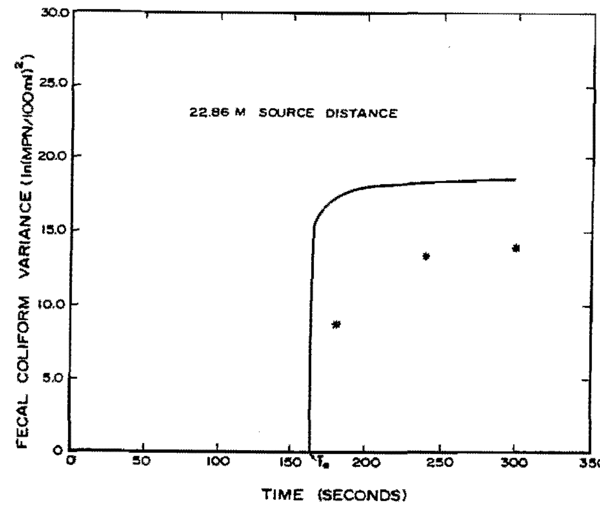

Figure 46. Observed and computed variances for 22.86 m (75 ft) source distance. Variances were computed assuming a Dirac delta initial distribution.

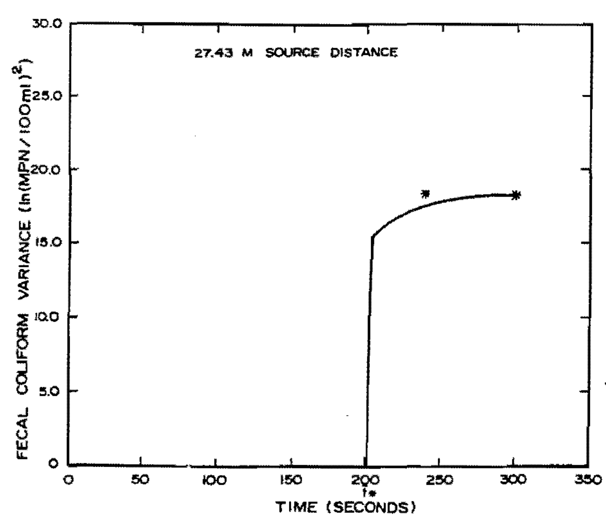

Figure 47. Observed and computed variances for 27.43 m (90 ft) source distance. Variances were computed assuming a Dirac delta initial distribution.

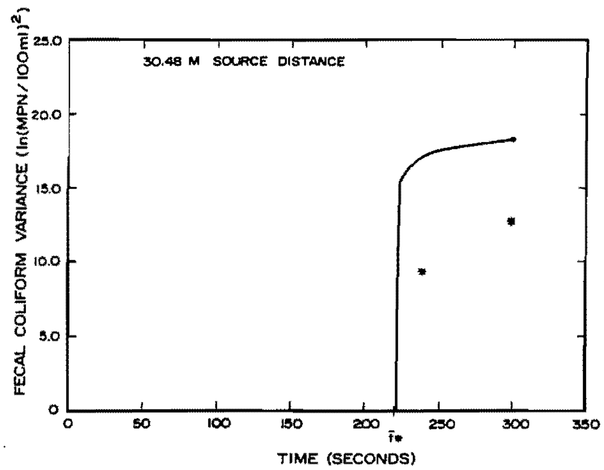

Figure 48. Observed and computed variances for 30.48 m (100 ft) source distance. Variances were computed assuming a Dirac delta initial distribution.

reached a relatively constant value early in the runoff period.

Probability Density Function

Regardless of whether a Dirac delta initial distribution or a normal initial distribution is assumed, the resulting pdf is normal at the outlet. This was the result of assuming independent Gaussian white noise terms and solving a linear, constant coefficient differential equation. Generally, bacterial counts are assumed to be log normally distributed, that is, sample distributions have been best described by the log normal distribution. Examination of the data for this study would explain this reasoning. It is impossible to have bacterial counts less than zero, and the large variance exhibited by these data would make the probability very high that a value less than zero could be obtained if the distribution was assumed to be normal.

For either initial distribution, Dirac delta or normal, the mean of the pdf at the slope outlet is the same because the input value for the constant in Dirac delta and the mean of normal initial distribution were assumed to be the same value. The same does not hold true for the variances. Recall from Equations 53 and 55 that the variances for the joint pdf at the slope outlet were $G(t;k)$ and $[t-t^*/pt]\ V + G(t;k)$ for Dirac delta and normal initial distributions, respectively. Obviously, the variance at the slope outlet is larger if a normal distribution was initially assumed. When the distribution was derived, V was assumed to be the variance of the initial distribution. If V is set to zero, the two initial distributions result in the same pdf at the outlet. A value for V can be assumed, but this was not considered when determining parameters to describe the variance in the preceding section.

There were insufficient observations at any source distance and time to verify the predicted distribution with the observed data. Figure 49 illustrates the cumulative distributions at t=90 sec and t=150 sec for the 3.04 m (10 ft) source distance. A constant standard deviation of 4000 MPN/100 ml was assumed in plotting the cumulative distributions. Also, the probability of attaining an MPN value of 2000 MPN/100 ml, the recreational water standard, for these distributions is given in Figure 49. This figure indicates how the pdf may be used to establish probabilities that pollution standards may be exceeded.

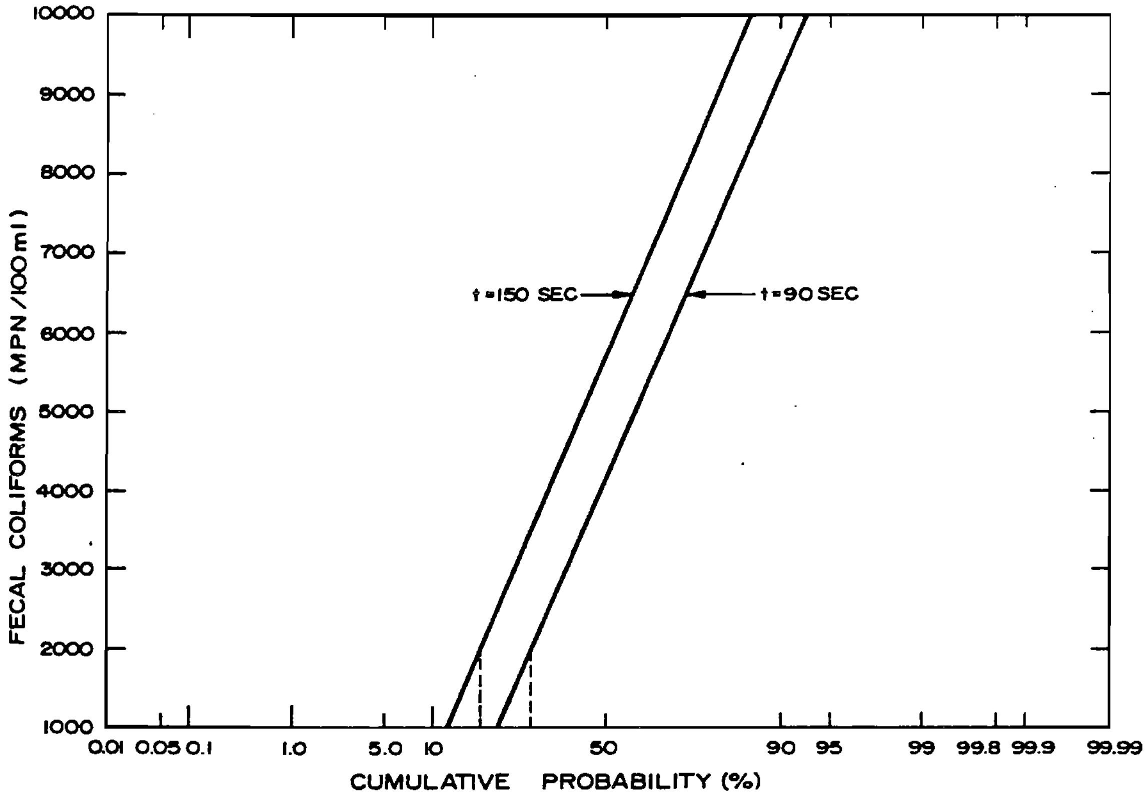

Figure 49. Comparison of cumulative density functions for t=90 sec and t = 150 sec for the observed means for the 3.04 m (10 ft) source distance with an assumed standard deviation of 4000 MPN/100 ml.

DISCUSSION

Model Applicability

Smooth Surfaces

The models used to represent runoff and coliform bacteria concentrations incorporated randomness in the initial conditions and nonhomogeneous terms. The randomness represented the wide variation in the data and the lack of precision of the MPN method of determining coliform counts.

The validity of the kinematic wave approximation for overland flow modeling has been demonstrated (Kibler and Woolhiser 1970, Rovey et al. 1977). The significant validity question is on the deterministic form of the pollutant model (Equation 22). The model assumed that the pollutant moved at the velocity of the water. This assumption has been used in modeling other pollutants, including sediment (Hjelmfelt 1976) and salinity (Ingram and Woolhiser 1980). Brazil et al. (1979) utilized this assumption and a conservative tracer to derive estimates of the slope-roughness parameter (Chezy C) in the kinematic wave equation and validated their results. They concluded that the most important factors affecting travel time were rainfall intensity and source distance. The inability to continuously sample for fecal coliforms did not allow a similar effort to be undertaken here or a comparison of actual versus predicted travel times from the derived parameters. Comparison of the predicted travel times for water with the measured time at which bacteria counts first exceeded a value of 100 MPN/100 ml for the 12.19 m (40 ft), 15.24 m (50 ft), 27.43 m (90 ft), and 30.48 m (100 ft) source distances confirm the assumption that the bacteria travel at about the same velocity as the water (Table 13).

The formulation of the random pollutant model had the assumption that the Gaussian white noise terms were independent. There was no information on which to base this assumption. It definitely made the solution process much easier, but this assumption was not necessary to solve the equation.

The initial conditions and inputs were considered to be the random parts of the pollutant transport process. The velocity coefficient (Equation 26) could have been varied stochastically, as it was by Molyneux and Witten (1980), to represent spatial variation in the roughness term. Equation 26 with random coefficients would be much more complicated to solve and validate. Because of this and the consideration given above, the velocity component was considered deterministic throughout the analyses.

The use of constants for the coliform input, loss, and variance coefficients was discussed in the previous section. Coliform release data suggest that the input is essentially constant after 5 min. It is highly unlikely that the coliform counts were at these high levels initially. More likely, the erosion process achieved a steady state detachment within the 5 min initial period.

To provide this sort of variability in the model input, a linear piecewise continuous function could be utilized along the lines of the following

$$c = at_c \qquad t \leq t_c \quad . \; . \; . \quad (61)$$

$$c = a \qquad t \geq t_c \quad . \; . \; . \quad (62)$$

Table 13. Travel times of individual runs from 12.19, 15.24, 27.43, and 30.48 m (40, 50, 90, and 100 ft) source distances and interval in which fecal coliform count was greater than 100 MPN/100 ml.

Source Distance (m)	Travel Time[a] (sec)	Interval ($FC \geq 100$ MPN/100 ml) (sec)
12.19	100.62	150
12.19	111.97	150
12.19	107.33	150
12.19	104.94	150
12.19	98.88	150
15.24	122.45	150
15.24	109.32	150
15.24	145.01	180
15.24	133.94	180
15.24	139.01	150
27.43	225.54	180
27.43	232.07	240
27.43	223.81	240
27.43	225.97	180
27.43	235.39	180
30.48	198.18	240
30.48	243.94	240
30.48	227.13	240
30.48	223.69	300
30.48	224.58	240

[a]Calculated using precipitation and optimized roughness from Table 7.

where

t_c = time at which input is constant (T)

a = constant input value (MPN/100 ml * T)

A nonlinear function could be used in Equation 61 if desired, but present information has prevented use of either because a value for t_c is not known.

Data from Hendricks et al. (1979) indicated that bacterial adsorption over time could be described in the same form used for empirical infiltration equations. In the case of water infiltration into the soil, time is a surrogate variable for the decreasing gradients which results in the infiltration exhaustion phenomenon. For bacteria, the loss of attachment sites can be described by a similar surrogate time variable. The results of Hendricks et al. (1979) indicated the bacteria in the water column approached an equilibrium value. The same behavior was observed above as the logarithmic counts approached steady state.

The most promising technique to account for bacterial adsorption are the different isothermal relationships proposed by Hendricks et al. (1979) and Reddy et al. (1981). Problems in using these are still the same, such as relating the retention to a soil or hydraulic feature. Another consideration is how much of the total bacterial load is in contact with the surface. If an isotherm such as the linear Freundlich (Equation 58) proposed by Reddy et al. (1981) were substituted for r in Equation 22, the resulting equation would be

$$\frac{\partial c}{\partial t} + u\frac{\partial c}{\partial x} = -(q + K)\ \frac{c}{h} + \frac{b}{h} \quad . \qquad (63)$$

where

K = retention coefficient (L/T)

Another advantage of using the isothermal relationships would be the ability to handle the equilibrium condition.

A problem existed in determining the number of bacteria released into the flow. Khaleel et al. (1979) developed a model which estimated erosion rates for fecal material using an USLE-type relation. Estimates of an erodibility factor, k, for several different types of fecal material were presented by Khaleel et al. (1979), but it was still necessary to estimate the number of bacteria attached to an eroded particle.

Another consideration is the effect of rainfall intensity. A higher intensity with higher kinetic energy would erode more material and carry more bacteria with the flow. If this is an effect, it could have influenced the results of this study, since rainfall intensity did vary slightly over the experimental period (Table 7).

Application to the Soil Surface

Rangelands are not composed of concrete surfaces. Application of the above model to real soil surfaces requires estimation of appropriate parameter values. The response of a soil surface, even with constant rainfall intensity, poses a far more difficult problem in estimating hydraulic parameters, particularly the slope roughness parameter in the kinematic wave equations.

The biggest problem in determining coliform transport and impacts in a range system may be the detection of impacts given the background counts. This can be readily seen from the simulations made for the soil surface portion of the study (Appendix Tables 26-30). If one were to assume that the background distribution of fecal coliforms was stationary, then the control run of 19 June and the control run of 26 June could be used to determine the mean and variance of the normal distribution. Table 14 has both means and variances from these runs and note the means are not the same.

The quality standard for fecal coliforms suggested by APHA for recreational water is 2000 MPN/100 ml. Using the above means, there is a probability greater than 50 percent that coliform concentrations at the outflow may exceed the established standard. However, these statistics consider only the background and only one situation represented by the soil surface. Also, once the bacteria are in the channel, they may settle with sediment on attach themselves to the banks.

To examine the effect of the background on the detection of impacts, Figure 50 compares the MPN values over time for the initial control run and three runs with source material present.

Table 14. Means and variances for log-transformed fecal coliform counts from simulations on 19 June 1981 before source material was introduced, and 26 June 1981 after simulations with source material were completed.

Date	Mean (MPN/100 ml)	Variance $(\text{MPN}/100\ \text{ml})^2$
19 June	22950	$3.95 * 10^8$
26 June	4833	$2.25 * 10^6$

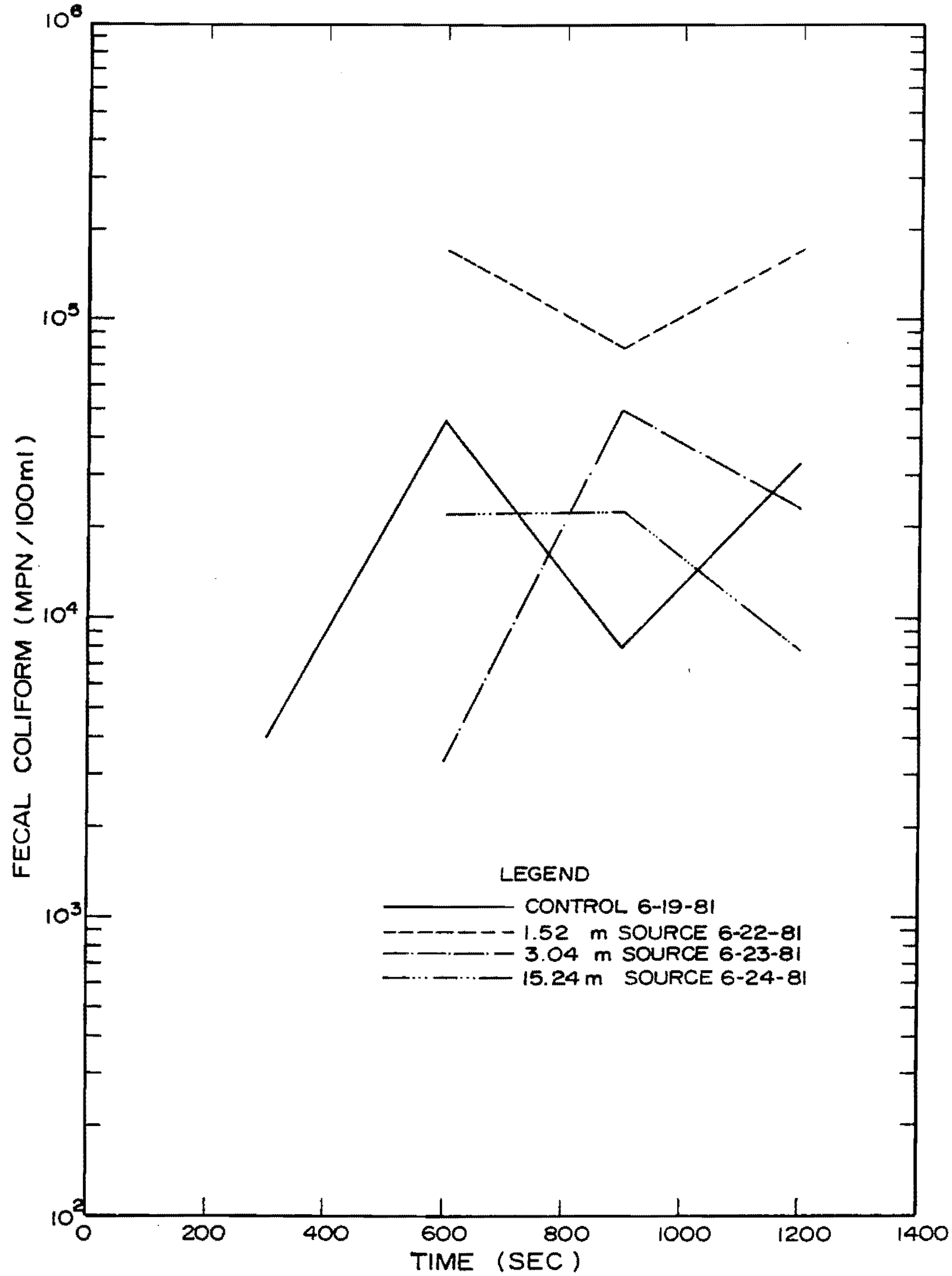

Figure 50. Observed fecal coliform counts from control run prior to application of source material and three runs with source material on soil surface.

Only the simulation with the source material 1.27 m (5 ft) above the outlet was consistently above the control run. The counts for the 15.24 m (50 ft) source distance showed no peak that could be associated with arrival of the source material at the outlet.

As for the concrete runoff surface, the most difficult parameter to estimate is the coliform retention parameter if the background distribution is assumed to be stationary and steady-state. Hopefully, the retention could have been tied to the roughness parameter in the overland flow equations, but from previous efforts on the concrete surface, it appears this parameter may be constant for a range of roughnesses. In addition, there was to be no replication on the soil surface as there was on the concrete because the surfaces which would have allowed a least squares fitting of this parameter. To complete the modeling exercise, the retention of highest fit was obtained from the 1.52 m (5 ft) source distance and this applied to the 3.04 m (10 ft) and 15.24 m (50 ft) source distances.

Rainfall Excess Modeling

A soil surface introduces a need to account for losses in converting rainfall to rainfall excess and this requires more complex modeling than was necessary for the concrete plot. Also, declining losses or relatively increasing values of rainfall excess change the method of solution of the overland flow equations. By considering the rainfall excess hyetograph as a histogram, the method of characteristics may be applied to a single overland flow plane (Eggert 1976), and that approach was chosen here.

Rainfall excess was estimated by the Green and Ampt (1911) infiltration equation. This approach has been shown to give reasonably accurate measurements of infiltration under different upper boundary conditions, and its parameters are physically based. Application of the equation has been hindered by parameter estimation, and the implicit nature of the equation. Recently an explicit approximation derived by Li et al. (1976) was found to be about 8 percent in error when compared to the solution of the implicit equation and has increased the utility of the Green and Ampt equation.

Li et al. (1976) approximated calculation of the change in cumulative infiltration over a discrete time interval by

$$\Delta F(t+\Delta t) = \frac{[(2F(t)-K_o\Delta t)^2 + 8K_o\Delta t(A+F(t))]^{\frac{1}{2}}}{2} - \frac{(2F(t)-K_o\Delta t)}{2} \quad . \; . \; . \quad (64)$$

where

ΔF = change in cumulative infiltration for period ($t + \Delta t$) (L)
$F(t)$ = cumulative infiltration at time t (L)
K_o = effective hydraulic conductivity (L/T)
A = S_{av} (θs - θi)
S_{av} = average suction head (L)
θs = saturated water content
θi = initial water content

The infiltration rate is calculated for the period t to t + Δt by

$$f(t+\Delta t) = \frac{\Delta F(t+\Delta t)}{(t+\Delta t - t)} \quad . \; . \quad (65)$$

where

$f(t + \Delta t)$ = infiltration rate (L/T)

The two parameters required for Equation 64 are the effective hydraulic conductivity (K_o) and the average suction head at the wetting front (S_{av}). A value for S_{av} was obtained from a relationship between soil texture classes and this parameter provided by Brakensiek (personal communication, 1981). The soil was in the silty clay class with a S_{av} of 29.22 cm (11.50 in). The hydraulic conductivity was estimated from the essentially constant portion of the hydrograph for the control run of 19 June. The difference between the calibrated rainfall rate and runoff rate was found to be 0.0223 cm/min (0.53 in/hr). This difference could be used as the hydraulic conductivity measure, but Smith (1976) showed bias was introduced by overland flow routing on infiltration measurements by a 1.83 m (6 ft) long type F infiltrometer plot. For the 30.48 m (100 ft) long plot used in this study, the bias was expected to be greater. Therefore, the conductivity value was set at 0.015 cm/min (0.35 in/hr). Values for porosity and initial moisture content were not known, but it can be seen in Equation 64 that it is the difference of these values which is important. All runs were conducted approximately 24 hrs after 20 min of simulated rainfall on the previous day. Since the soil mass was still quite moist after only a day of evaporation and drainage, the difference between θ_s and θ_i was assumed to be very small and set at 0.01.

Solution to Overland Flow Equations

The kinematic wave equations (Equations 3 and 6) are solved in the same manner used for the steady input case. Equation 15 is used to solve for the limiting characteristic as before, but by looking at Figure 51, the difference in the depth calculation can be seen. The change in depth for the period from t to t + Δt is

$$h(t + \Delta t) = h(t) + \Delta t\, p(t + \Delta t) \qquad (66)$$

where

$h(t + \Delta t)$ = depth at time t + Δt (L)

$h(t)$ = depth at time t (L)

$p(t + \Delta t)$ = precipitation excess for period t + Δt (L)

The disturbance will propagate downslope at a rate equaled to that given by Equation 15

$$\frac{dx}{dt} = amh^{m-1} \quad . \; . \; . \; . \; . \; . \; . \quad (15)$$

By substituting Equation 66 into Equation 15 for h, the downslope distance moved by the characteristic over a period of constant rainfall excess is

$$x(t + \Delta t) = am \int_0^{\Delta t} (h(t) + sp(t + \Delta t))^{m-1}\, ds + x(t)$$

$$= \frac{a}{p(t + \Delta t)} \left((h(t) + \Delta t p(t + \Delta t))^m - (h(t))^m\right) + x(t) \quad . \; . \; . \; . \quad (67)$$

where

$x(t)$ = distance downslope at time t (L)

$x(t+\Delta t)$ = distance downslope at time t + Δt (L)

s = dummy integration variable

This equation is solved iteratively until x(t + Δt) is greater than or equal to the slope length L. The time at which the limiting characteristic reaches the bottom of the slope is termed the time of concentration and is analogous to the equilibrium time in the steady input case. By rearranging Equation 67, the time of concentration becomes

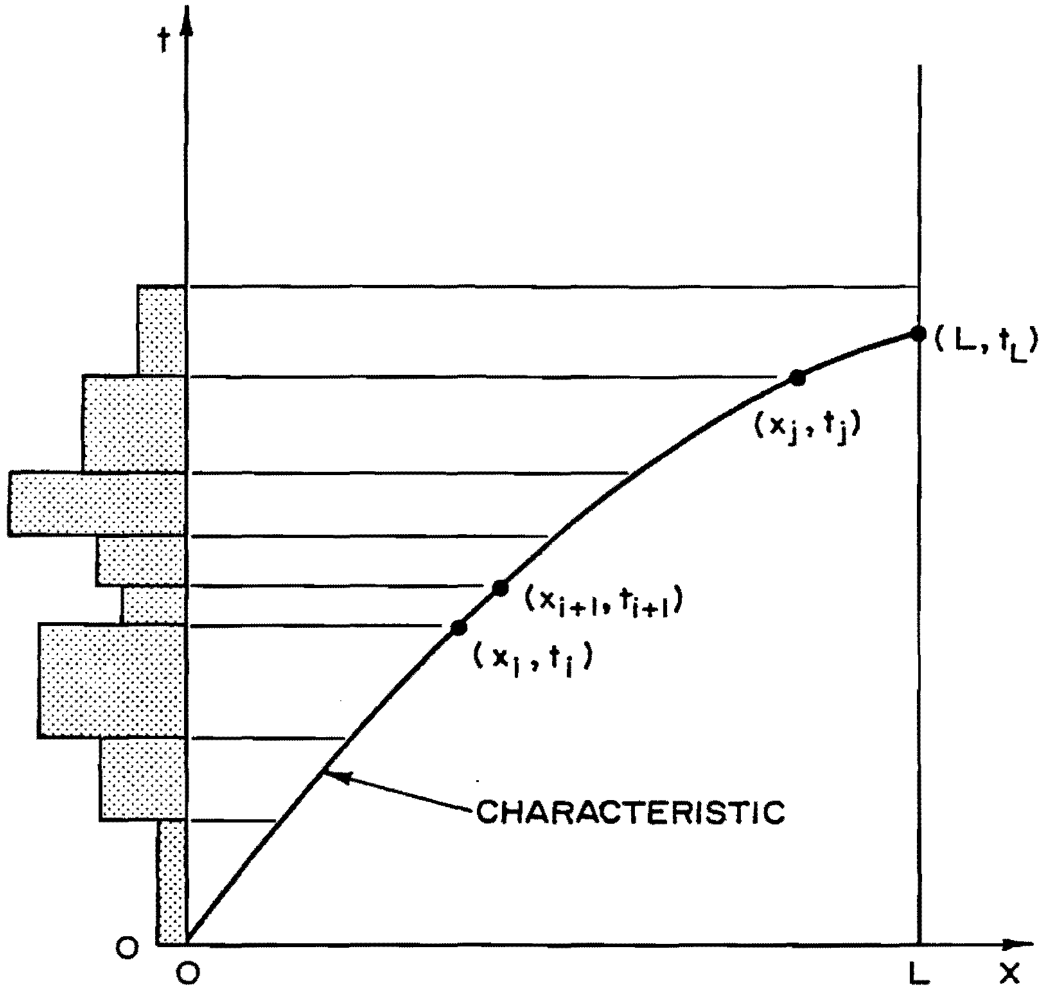

Figure 51. Solution domain for method of characteristics with unsteady input (from Eggert 1976).

$$\Delta t_c = \left(\left[\frac{L-x(t)p(t+\Delta t)}{a} + (h(t))^m\right]^{1/m} - h(t)\right) \Big/ p(t+\Delta t) \quad . \quad . \quad . \quad (68a)$$

$$t_c = t + \Delta tc \quad . \quad . \quad . \quad . \quad . \quad . \quad (68b)$$

where

t_c = time of concentration

For solution of the colifiorm transport model, travel time (t^*) is required. This is solved in the same manner as the limiting characteristic except Equations 39a and b are used. The distance traveled assuming particle velocity is

$$x(t+\Delta t) = \frac{a}{mp(t+\Delta t)} [(h(t) + \Delta t\ p(t+\Delta t))^m - (h(t))^m] + x(t) \quad . \quad . \quad . \quad . \quad (69)$$

Equation 68 is solved successively until the slope outlet has been reached and Equation 68 is rearranged to solve for travel time

$$\Delta t^* = \left(\left[\frac{(L - x(t)) + p(t + \Delta t)}{a} + (h(t))^m \right]^{1/m} - h(t) \right) / p(t + \Delta t) \quad . \quad . \quad . \quad . \quad (70a)$$

$$t^* = t + \Delta t^* \quad . \quad . \quad . \quad . \quad . \quad (70b)$$

As was done for the concrete runoff surface, the slope-roughness parameter was optimized for each simulation. A correction for flume storage was applied, but no correction was made for the sheet metal transition. Optimization was done with the equation describing the limiting characteristic since pollutant travel times were not known. Optimized values are in Table 15. The values for Chezy C reported in Table 11 were derived assuming a slope of 0.06. The optimized values were higher (greater roughness) than those reported by Rovey et al. (1977) for a bare clay-loam soil which had been eroded. The range indicated by Rovey et al. was 198.75 - 83.33 $cm^{1/2}/sec$ while the optimized values ranged from 59.2 - 31.0 $cm^{1/2}/sec$. The soil surface in this study was never eroded and thus had a lower roughness. Also, any errors in the rainfall excess determination included would influence the result.

Another simplification in the model was that the rainfall excess and the overland flow equations are considered spatially homogeneous. In actuality the spatial distribution of rainfall excess is known to affect overland flow hydrographs as shown by Smith and Hebbert (1979). There was insufficient detail to incorporate these features into this effort.

Coliform Model

Naturally, Equation 26 would be the preferred model for the simulation, but again there is insufficient spatial detail to use this equation. The approach was taken to model the mean since there was a lack of replication which would be needed to provide estimates of variance terms.

Only changes in the mean with time were modeled, and the background

Table 15. Optimized slope-roughness factors and calculated travel and concentration times for soil surface simulations.

Source Distance (m)	Alpha ($cm^{1/2}/sec$)	Chezy C[a] ($cm^{1/2}/sec$)	R^2[b]	t_c (sec)	t^* (sec)
Control	8.6	35.1	0.77	731.5	
1.524	14.5	59.2	0.74	934.8	418.3
3.048	11.0	44.9	0.70	1051.5	525.8
15.240	7.6	31.0	0.91	995.7	877.9
Control	11.1	45.3	0.40	1047.3	

[a]Calculated from ALPHA = C $\sqrt{S_o}$ where S_o is land slope surface (0.06 for this case).

[b]From observed values for the rising graph, at least three observations were used.

mean count was assumed constant and added to the equation. The following equation describes the change in the mean

$$\frac{d\bar{c}}{dt} = \frac{p(t)}{h(t)}\bar{c} + \frac{b}{h(t)} - \frac{r}{h(t)} - \frac{\bar{c}_o p(t)}{h(t)} \quad . \quad . \quad . \quad . \quad (71)$$

where

$p(t)$	=	rainfall excess at time t (L/T)
$h(t)$	=	depth of flow at time t (L)
b,r	=	source-sink terms
$\bar{c}_o$	=	mean of background counts (MPN/100 ml

Note that this is Equation 37 with the background added. Also, when b is equal to zero, r is equal to zero, the equation describes a steady value of $\bar{c}_o$. The values for rainfall excess (p(t)) and depth (h(t)) are subscripted to indicate a piecewise representation rather than the continuous variation used in previous efforts. This equation can use real time rather than travel time as an initial condition. If real time is used, the values of b and r are set to zero until travel time is attained. The initial condition becomes

$$\bar{c}(o) = 0 \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (72)$$

The value used for b in the concrete surface case was used for this case also, therefore b + 6 * 10^6 MPN/100 ml. The background mean was taken from Table 14 for the control run of 19 June which was 23000 MPN/100 ml. Only a value of r remains to be determined.

The discrete representation of rainfall excess prevented a direct solution of the sort used for coliform movement over the concrete surface. For the soil surface, Equation 68 was solved by a fourth order Runge-Kutta method (Atkinson 1978). This technique solves equations of the form

$$\frac{dy}{dx} = f(x,y) \quad . \quad . \quad . \quad . \quad . \quad (73)$$

With the algorithm

$$Y(N + 1) = Y(N) + \Delta x/6(V1 + 2*V2 + 2*V3 + V4) \quad (74)$$

where

Y(N + 1)	=	value of Y at next step
Y(N)	=	value of Y at present step
Δx	=	step size
V1	=	f(x(N), Y(N))
V2	=	f(x(N), + Δx/2, Y(N) + Δx*0.5*V1)
V3	=	f(x(N) + 0.5* Δx, Y(N) + Δx*0.5*V2)
V4	=	f(x(N) + Δx, Y(N) + Δx*V3)

Effort has been made to avoid a numerical solution throughout this study because it was felt this would decrease the utility of the model. The solution to an ordinary differential equation such as Equation 71 is simpler than the solution to a partial differential equation such as Equation 26. Only an initial condition is required for solution to the ordinary differential equation, whereas the partial differential equation would require initial and boundary conditions.

As indicated, the 22 June simulation was utilized to determine a value for r which provided the highest R^2 for the observed coliform counts. Travel time for this simulation was 418.3 sec (Table 15). The step distance utilized was 0.5 sec and the value of the retention parameter r was 5999970 MPN/100 ml.

Using the travel times from Table 15 and the rainfall excess data from Appendix Tables 22 and 23, the remaining source distances 3.04 (10 ft) and 15.24 m (50 ft) were simulated (Figures 52 and 53).

From Figures 52 and 53, it would appear that the means were reasonably

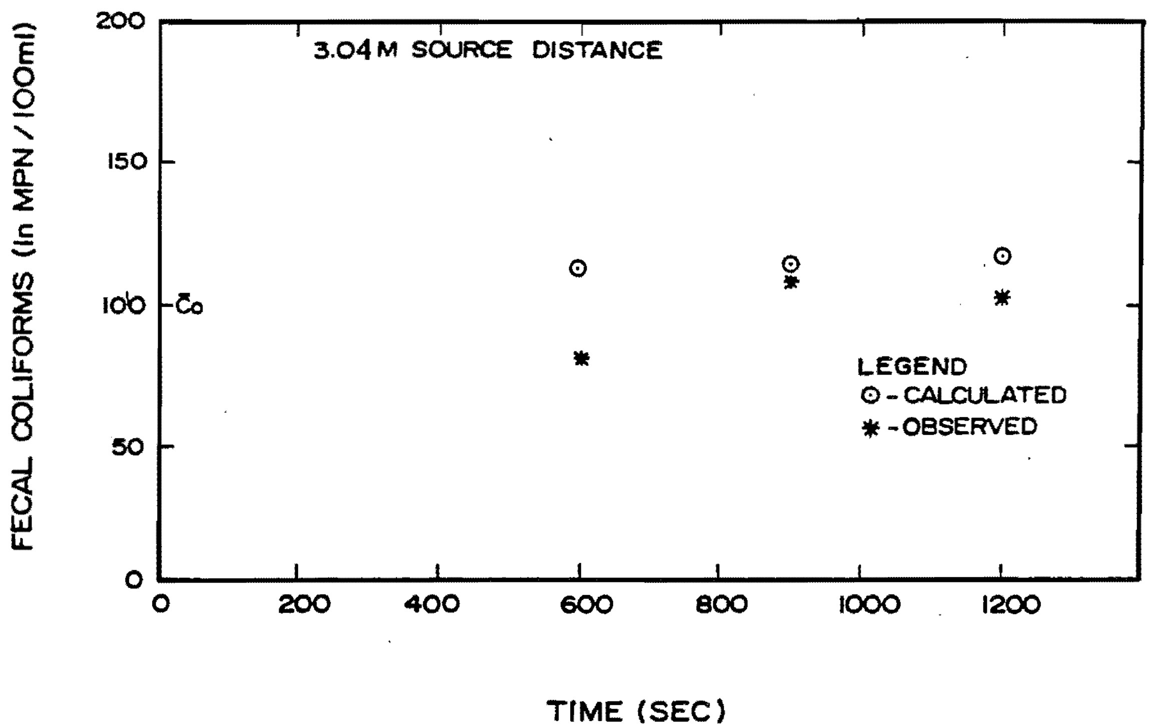

Figure 52. Observed and computed means for 3.04 m (10 ft) source distance on the soil surface.

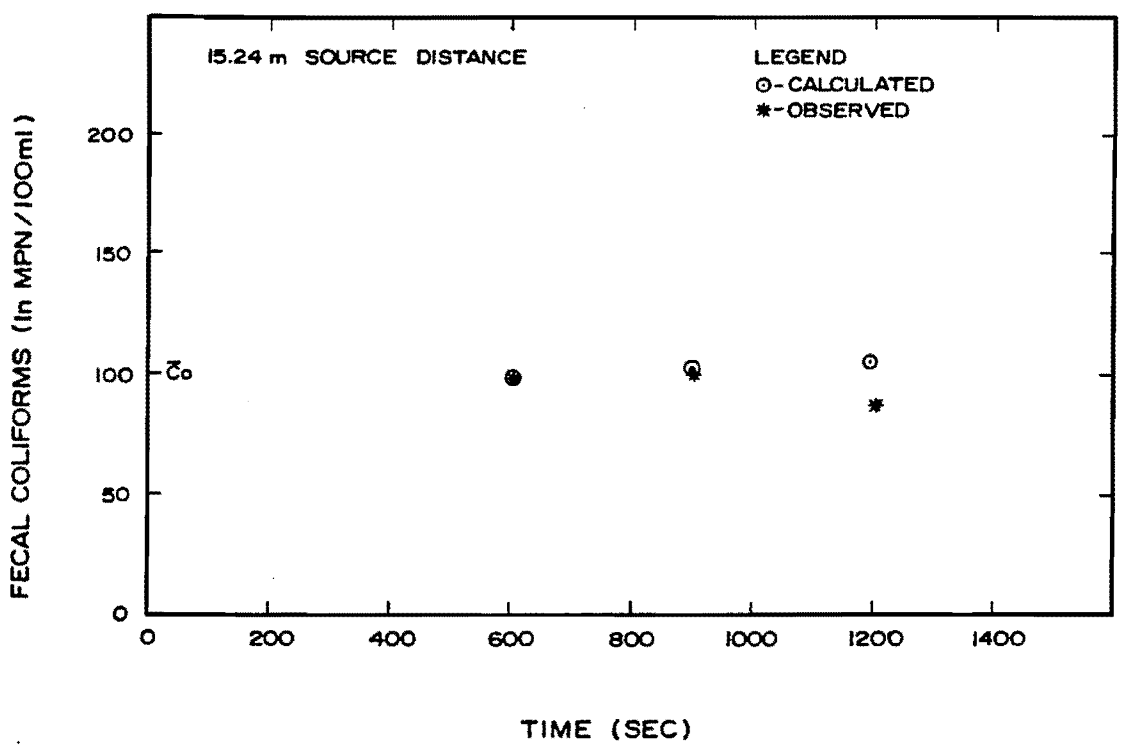

Figure 53. Observed and computed means for 15.24 m (50 ft) source distance on the soil surface.

simulated, but note that the ordinate scale is logarithmic. The high background mean count elevated values into the proper range. A simulation could have been conducted utilizing the background mean only, that is, setting b and r to zero, and the results would have been adequate. Obviously, the background counts are so high that the effect of a single source is insignificant. The impact of the background mean can be seen in Table 16. The observed and predicted counts are compared for the 3.04 m (10 ft) source distance. The first column of simulated values had a background mean count of 4000 MPN/100 ml, and the second column of simulated values had a background mean of 23000 MPN/100 ml. Obviously, simulated values from both background counts do not appear to stimulate the observed values very well. What the high background observed on the 19 June run has done was to create an artificially high value which made simulation meaningless. Also, the fact that the observed values were from a single run and not a replication, hence they were not means, must be remembered.

The soil surface in this study was a disturbed state (transported in bulk to the plot; no vegetation) which is uncharacteristic of most rangeland systems. Large amounts of soil material were removed during each run and this certainly added to the confusing results.

The control run of 26 June (Table 14) provided more confounding evidence. Though the soil surface had been contaminated by the previous runs with fecal material on the surface, this run had relatively low counts. The narrow range in the counts for this run (3300-6300 MPN/100 ml) was more expected for the control run prior to any fecal application while the control run of 19 June would have been anticipated following the runs with fecal material.

Table 16. Observed coliform counts and simulated means for the 3.04 m (10 ft) source distance assuming background mean counts of 4000 MPN/100 ml and 23000 MPN/100 ml.

Time (sec)	Fecal Coliforms (obs) (MPN/100 ml)	Fecal Coliforms (sim)[a] (MPN/100 ml)	Fecal Coliforms (sim)[b] (MPN/100 ml)
600	3300	53378	72568
900	49000	97505	116503
1200	23000	102854	121852

[a]Background mean is 4000 MPN/100 ml.

[b]Background mean is 23000 MPN/100 ml.

PART III

SUMMARY AND CONCLUSIONS

Domestic livestock grazing on rangelands are known to impact water quality by increasing the numbers of fecal indicator bacteria. By modeling movement into the stream of these indicator bacteria under a variety of conditions, the effects that various grazing practices may have on the water resource can be assessed and proper management utilized.

Studies were conducted during the summers of 1980 and 1981 to determine the peak FC release from cattle feces. A longevity treatment was conducted to determine how old fecal deposits must be before they no longer release significant numbers of FC. The longevity study was also used to develop a regression model to predict FC release with age. A recurrent-rainfall treatment was conducted to determine the effect of raining more than once on a fecal deposit. A dung-pile treatment was used to test the legitimacy of the grossly-manipulated fecal deposits used in the other treatments. Intensity treatments were conducted to determine the effect of rainfall intensity on FC release.

The fecal deposits, used in the six treatments, were of two types--natural dung piles and standard cowpies. A rainfall simulator was used to produce rainfall events. The runoff samples were primarily taken for peak FC counts. The FC counts were enumerated using the MPN method.

The following conclusions were reached:

1. The bacterial pollution potential of cattle fecal deposits is very great. Fecal deposits of less than 5 days of unrained-on age released fecal coliform concentrations on the order of millions per 100 ml. Fecal deposits as old as 30 days of unrained-on age produced concentrations on the order of 40,000 per 100 ml.

2. An equilibrium rate of fecal coliform release is reached after no more than 10 minutes of rainfall. This equilibrium relationship is characteristic of the latter stages of a typical bacterial growth curve. Proceeding from the steady state exponential phase within the bovine digestive system, the fecal coliforms pass through the retardation phase, maximum population phase, and the death phase, respectively. The retardation phase lasts about 1 day, the maximum population phase about 2 days, and the death phase continues beyond 30 days for an unknown length of time.

3. Hundred-day-old fecal deposits produce FC counts that exceed recreational water quality standards, but the release from these fecal deposits is much smaller than the release from 2-day-old fecal deposits.

4. The log-log regression, $\text{Log } Y = 7.57 - 1.97 \text{ Log } X$, was determined to be the most appropriate expression to use in predicting FC release from once-wet fecal deposits.

5. Recurrent rainfall reduces the peak FC below the peak release levels of once-wet fecal deposits; however, variable rewet data indicate that the relation is not entirely predictable.

6. The standard cowpie regression was not significantly different from the

natural dung-pile regression, so the cowpie regression can be used to determine the FC release from naturally-occurring fecal deposits.

7. Rainfall intensity is only significant after the fecal deposits are completely air dry, and then the lower the rainfall intensity, the later and higher the peak counts.

Watershed pollutant modeling is generally divided into two phases, upland and channel processes. This model considered only the upland phase, and since bacteria are not readily transported through porous media, overland flow was considered the primary vector.

To simplify initial, boundary, and hydraulic conditions, simulations were initially conducted on a smooth concrete surface with a single source. Justification for this approach was that if the simple situation could not be described, then how could the more difficult conditions be simulated.

The simulations on the concrete surface centered on determining the effect of distance from the source material to the outlet on coliform movement. Nine distances with five replications were used. Simulated rainfall was assumed constant for a run, but changed between simulations. All runs on the concrete surface achieved equilibrium flow conditions. All simulations lasted 10 min. The following season, the concrete plot was covered by a clay soil to a depth of 45.72 cm (18 in). Only five simulations were conducted, two control and three with source material at 1.52, 3.04, and 15.24 m (5, 10, and 50 ft, respectively).

Because it has been shown to be the most reliable indicator of health hazards, fecal coliforms were used. All analyses were conducted using the MPN method.

Overland flow was modeled using the kinematic wave approximation with the Chezy friction law assumed to describe the normal flow situation. On the soil surface, rainfall excess was generated by the Green and Ampt infiltration equation. For both surfaces, characteristic solutions were used.

Fecal coliform movement was described by a continuity equation which incorporated source-sink terms for background, input, and loss of coliforms. Given the highly variable nature of the coliform response, and the imprecision of the MPN method, the coliform transport equation was recast as a random differential equation by adding independent Gaussian white noise terms for space and time variability. Solution of this equation can be either in the form of the probability density function or the moments of the stochastic process.

The resulting probability density function was found to be normal whether the initial distribution was assumed to be a normal distribution or a constant, Dirac delta function. Thus, the mean and variance were required to fully describe the distribution. Lack of spatial information resulted in solution for the mean at the outlet over time. The solution for the concrete surface had three forms depending on the relationship between travel time and flow conditions.

Coliform release from fecal materials of cattle (previously mentioned) determined input values. On the concrete surface, background counts were not important since the plot was scrubbed with chlorine between simulations. The soil surface did have background counts that were considered part of the source term. The retention parameter from the mean equation and two variance terms had to be determined. The replications on the concrete surface allowed estimation of the parameters by least squares. The retention parameter appeared to be relatively constant for all cases, unsteady and steady state. Only one fit could be obtained for the variance terms. There were no replications on the soil surface so the variance was not modeled. The retention parameter was determined from 1.52 m (5 ft) source distance run and the background value from a control run conducted prior to application of fecal material.

For the concrete surface, the fits of observed to predicted values were poor, particularly for the early rising portion of the run. Simulation of this unsteady period had been considered important because unsteady characteristics dominate natural systems, but for this smooth surface, that steady state was achieved early in the event. The variances on the concrete surface appeared to follow much the same pattern, and the model failed to fit for much the same reasons. Simulations of the mean counts for the soil surface had to be by numerical solution to the ordinary differential equation describing the mean because of the nature of the solution for the overland flow hydraulics. The high background counts made any assessment of impacts of the source difficult.

Overall, the model did not do very well at the quantitative prediction of bacteria movement. Qualitatively, it was shown that on smooth surfaces, such as concrete, the bacteria can be moved long distances (30.48 m, 100 ft). The soil surface was more complex, and impacts were not as readily seen.

It is believed that the random approach used to describe the pollution problem posed in this study has the most utility. Since coliform counts do exhibit high variability and probability rather than absolute, statements about certain limits may have more meaning.

As with most first generation modeling efforts, there are some limitations with this model. The most overriding is that the model is applicable for the rising side of the hydrograph. The recession graph has not been tested. Obviously, relationships would change on the recession graph since there would be no rainfall to erode or remove the bacteria, and it is not known how overland flow by itself would influence coliform pickup.

The white noise terms in the development of the random differential equation were assumed to be independent. This probably was not a limiting assumption, but if the noise terms are correlated, another pdf would probably result from the solution of the differential equation.

The most obvious implication from this study in terms of range management considerations was the lack of impact of fecal material on coliform counts emanating from the soil surface. The background counts obtained in the initial control run were extremely high. If this is a natural condition, then grazing impacts may be minimal. Bacteria transport is also a function of the soil material on which the event is occurring. Soil materials such as those derived from Mancos shales, which are found in the Price River Basin of Utah, are easily eroded and the potential for bacterial transport over long distances is high. A sandy soil material with relatively high infiltration capacity would not be so easily eroded and potential to move bacteria would be low. Again, there is not much that is known and no final conclusions can be drawn from this study as to the relative nature of upland versus channel contributions to bacterial pollution of rangeland streams.

Future research on this topic will need to take several directions in order to understand and quantify the processes governing bacteria movement in overland flow. The following are suggested.

1. It would be advantageous to be able to use the partial differential equation to describe both space-time

relationships. The problems in using this technique have been alluded to in the text.

A technique to sample the overland flow on the flow surface could provide verifying points on the surface. An example of this in the case of salt flow can be seen in Ingram and Woolhiser (1980) in which they sampled the overland flow EC at the slope midpoint. Of course, that is not so easily done for bacteria since sterile instruments are required, but such an effort would help in verifying Equation 26.

2. The loss or adsorption relationship needs to be more physically based. The isothermal relationships investigated by Hendricks et al. (1979) or suggested by Reddy et al. (1981) are probably adequate. What is required is more intensive development of the retention parameters and correlating this parameter to something such as surface roughness.

An experimental approach to this problem can follow the same procedure as utilized by Wu et al. (1978) in a study of the effects of spatial variability of roughness on overland flow. They covered an asphalt surface with different densities of gravel. A similar approach could be taken with the concrete runoff surface used in this study. With a constant rainfall excess rate, analyses could be applied as was in the concrete surface studies for this project. By using different materials such as rocks, gravel, astro-turf, a wide range of roughnesses could be established and correlation with a retention parameter of best fit attempted.

3. Information is needed on the relative movements of bacteria by sediment and directly in the flow. This would be important not only for the upland phase, but also for the channel phase. This is most likely in the realm of the microbiologist rather than the engineer or hydrologist.

4. In terms of present application of the model, the most immediate need would be an idea of the background counts. Again, the need to use large plot rainfall simulators for such studies should be emphasized. If the backround counts are high, such as experienced in this study, then it may be difficult to detect grazing impacts.

5. Once modeling of a single source has succeeded, then multiple sources need to be considered. This will probably need to start with a simple surface and work to the soil surfaces as was done here.

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APPENDICES

Appendix A

Derivation of Continuity of Mass for Pollutant

The following derivation was based on Hjelmfelt (1976) derivation. See Figure 54 for definitions.

$$ch\Delta x = \left[qc - \frac{\partial(qc)}{\partial x}\ \frac{\Delta x}{2}\right]\Delta t - \left[qc + \frac{\partial(qc)}{\partial x}\ \frac{\Delta x}{2}\right]\Delta t \pm S\Delta x\Delta t \quad . \ . \ . \ . \ . \quad \text{(A-1)}$$

where:

c	=	concentration of pollutant (M/L3)
q	=	flow per unit width (L^3/ T*L)
h	=	depth (L)
S	=	source-sink terms

$$\frac{ch}{\Delta t}\ \Delta x = \left[qc - \frac{\partial(qc)}{\partial x}\ \frac{\Delta x}{2}\right] - \left[qc - \frac{\partial(qc)}{\partial x}\ \frac{\Delta x}{2}\right] \pm S\Delta x \quad . \ . \ . \ . \ . \quad \text{(A-2)}$$

$$\lim_{t\to 0}\left[\frac{ch}{\Delta t}\right] = \lim_{t\to 0}\left[-\ \frac{\partial(qc)}{\partial x} \pm S\right] \quad . \ . \ . \quad \text{(A-3)}$$

$$\frac{\partial(ch)}{\partial t} = \frac{\partial(qc)}{\partial x} \pm S \quad . \ . \ . \quad \text{(A-4)}$$

$$c\frac{\partial h}{\partial t} + h\frac{\partial c}{\partial t} = -\ q\frac{\partial c}{\partial x} - c\frac{\partial q}{\partial x} \pm S \quad . \ . \ . \quad \text{(A-5)}$$

$$h\frac{\partial c}{\partial t} + q\frac{\partial c}{\partial x} = -\ c\left(\frac{\partial h}{\partial t} + \frac{\partial q}{\partial x}\right) \pm S \quad . \ . \ . \quad \text{(A-6)}$$

From Equation 3, it can be seen that the term in brackets on the right hand side of Equation A-6 in the lateral inflow. Assuming an impervious surface.

$$h\frac{\partial c}{\partial t} + q\frac{\partial c}{\partial x} = -\ cp \pm S \quad . \quad \text{(A-7)}$$

$$\frac{\partial c}{\partial t} + u\frac{\partial c}{\partial x} = -\ \frac{p}{h}\ c \pm \frac{S}{h} \quad . \ . \quad \text{(A-8)}$$

The source-sink term can be broken into the input and loss terms which are shown in Figure 54.

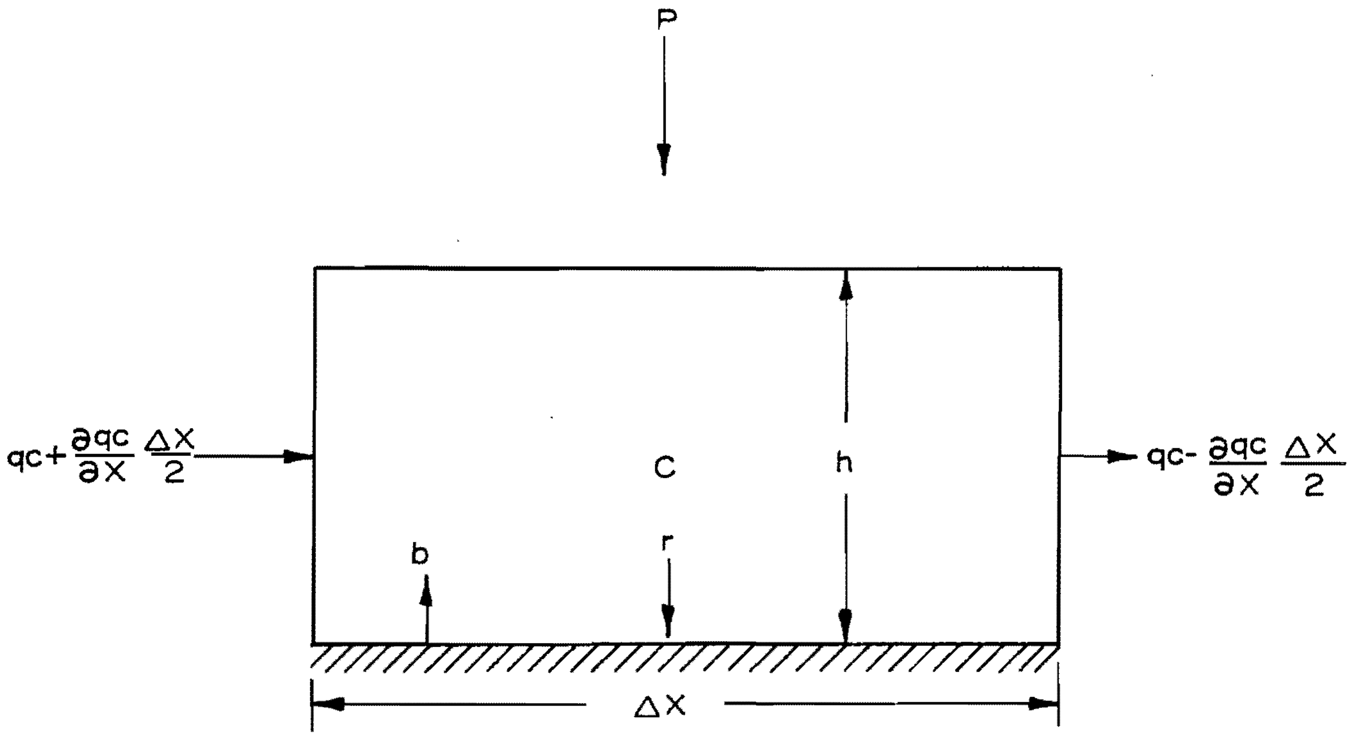

Figure 54. Control volume used to derive continuity of mass equation for pollutant flow.

Appendix B

Derivation of Density for Case 1

Assuming a Normal Initial Density

From equation 35, we can write the joint density as follows

$$f(c,t,b;k) = \frac{1}{2\pi(GV)^{\frac{1}{2}}} \exp \left[-\frac{1}{2}\left[\frac{(c-\bar{c})^2}{G} + \frac{(b-\bar{b})^2}{V}\right]\right] \quad \ldots . \text{(B-1)}$$

$$f(c,t;k) = \int_{-\infty}^{\infty} \frac{1}{2\pi(GV)^{\frac{1}{2}}} \exp \left[-\frac{1}{2}\left[\frac{(c-\bar{c})^2}{G} + \frac{(b-\bar{b})^2}{V}\right]\right] db \quad \ldots . \text{(B-2)}$$

Simplifying and making the following subsitutions

$$\bar{c} = \bar{b}H - rH$$

$$H = \frac{t-t^*}{pt}$$

$$\frac{1}{(2\pi(H^2V+G))^{\frac{1}{2}}} \exp\left[-\frac{H^2V+G}{2GV}\left(\frac{c^2V+2cVrH+\bar{b}G+r^2H^2V}{H^2V+G} - \left[\frac{cHV-rH^2V+\bar{b}G}{H^2V+G}\right]^2\right)\right] * \int_{-\infty}^{\infty} \frac{(H^2V+G)^{\frac{1}{2}}}{(2\pi GV)^{\frac{1}{2}}} \exp \left[-\frac{H^2V+B}{2GV}\left(b - \frac{CHV+rH^2V+\bar{b}G}{H^2V+G}\right)^2\right] db \quad \ldots . \text{(B-3)}$$

Integrating, simplifying, and completing the square

$$f(c,t;k) = \frac{1}{(2\pi(H^2V+B))^{\frac{1}{2}}} \exp \left[-\frac{1}{2(H^2V+G)} (c-H(\bar{b}-r))^2\right] \quad \ldots . \text{(B-4)}$$

$$f(c,t;k) \sim N(H(\bar{b}-r);\ (H^2V+G)) \quad \ldots . \text{(B-5)}$$

Appendix C

Data

Table 17. Data for 1.52 m (5 ft) source distance from the concrete runoff surface.

Time (sec)	Flow (cm^3/sec)	Corrected Flow (cm^3/sec)	Fecal Coliform (MPN/100 ml)	Total Coliform (MPN/100 ml)
		27 June, 1980		
90	40.431	138.470	330	300
120	336.205	362.940	490	1700
150	595.440	595.440	790	2800
180	595.440	615.335	1400	1800
240	615.335	615.335	1600	3300
300	615.335	615.335	4900	4900
		29 June, 1980		
90	0.050	7.180	13	49
120	81.002	195.080	330	330
150	289.756	257.670	4900	4900
180	467.176	542.040	7900	7900
240	677.444	677.444	3300	3300
300	677.444	677.444	4900	4900
		1 July, 1980		
90	144.053	304.480	23	23
120	433.975	433.975	700	700
150	519.818	519.818	490	490
180	519.818	519.818	490	490
240	519.818	519.818	490	490
300	519.818	519.818	460	490
		2 July, 1980		
90	69.896	191.100	330	330
120	467.176	477.870	24000	24000
150	556.843	556.843	3300	3300
180	556.843	595.440	7000	7000
240	656.334	666.838	2800	2800
300	666.838	666.838	3300	3300

Table 18. Data for 3.04 m (10 ft) source distance from the concrete runoff surface.

Time (sec)	Flow (cm^3/sec)	Corrected Flow (cm^3/sec)	Fecal Coliform (MPN/100 ml)	Total Coliform (MPN/100 ml)
		3 July, 1980		
90	89.998	223.680	49	79
120	538.136	538.136	24000	24000
150	656.334	677.444	13000	13000
180	677.444	709.880	7900	7900
240	720.899	720.899	3300	3300
300	720.899	720.899	7900	7900
		4 July, 1980		
90	106.304	248.900	33	49
120	501.888	595.440	330	700
150	595.440	595.440	940	1400
180	595.440	595.440	790	790
240	595.440	595.440	1700	1700
300	595.440	595.440	1700	1700
		21 July, 1980		
90	10.823	69.640	110	170
120	198.960	263.130	54000	54000
150	322.501	322.501	22000	22000
180	343.188	357.419	11000	11000
240	357.419	357.419	7900	7900
300	357.419	357.419	14000	14000
		22 July, 1980		
90	48.459	153.620	130	130
120	302.597	306.160	1300	1300
150	402.260	402.260	1300	1300
180	402.260	402.260	1300	1300
240	402.260	402.260	490	490
300	379.434	402.260	1300	1300
		26 July, 1980		
90	16.654	86.170	230	230
120	198.960	241.740	7900	7900
150	315.780	357.419	7900	13000
180	357.419	357.419	14000	14000
240	357.419	357.419	9400	9400
300	357.419	357.419	17000	17000

Table 19. Data for 6.10 m (20 ft) source distance from the concrete runoff surface.

Time (sec)	Flow (cm^3/sec)	Corrected Flow (cm^3/sec)	Fecal Coliform (MPN/100 ml)	Total Coliform (MPN/100 ml)
		5 July, 1980		
90	113.295	259.460	0	0
120	556.843	556.843	110	110
150	635.632	635.632	110	110
180	635.632	615.335	130	130
240	615.335	615.335	630	630
300	615.335	615.335	460	460
		7 July, 1980		
90	152.458	316.450	0	2
120	467.176	467.176	1800	1800
150	605.337	615.335	4600	4600
180	615.335	615.335	7900	7900
240	615.335	615.335	7900	7900
300	615.335	615.335	7900	7900
		10 July, 1980		
90	116.896	264.840	17	17
120	450.388	450.388	1300	1300
150	595.440	595.440	11000	11000
180	595.440	595.440	13000	13000
240	595.440	595.440	3300	3300
300	595.440	595.440	2300	2300
		27 July, 1980		
90	24.092	104.300	2	2
120	241.752	277.400	330	330
150	343.188	357.419	1100	1100
180	357.419	357.419	2200	4900
240	357.419	357.419	1700	1700
300	357.419	357.419	490	490
		28 July, 1980		
90	4.264	45.260	13	33
120	161.156	246.710	2300	2300
150	315.780	343.188	7900	7900
180	343.188	357.419	4900	4900
240	357.419	343.188	1300	1300
300	343.188	343.188	1300	1300

Table 20. Data for 9.14 m (30 ft) source distance from the concrete runoff surface.

Time (sec)	Flow (cm^3/sec)	Corrected Flow (cm^3/sec)	Fecal Coliform (MPN/100 ml)	Total Coliform (MPN/100 ml)
		29 July, 1980		
90	34.984	127.670	2	4
120	230.574	237.70	49	49
150	336.205	336.205	3300	3300
180	350.260	350.260	4900	7900
240	350.260	350.260	3300	4900
300	350.260	350.260	4900	4900
		30 July, 1980		
90	42.355	142.170	2	2
120	302.597	316.860	170	170
150	386.952	402.260	1300	1300
180	402.260	402.260	2200	2200
240	402.260	402.260	3300	3300
300	402.260	402.260	1800	1800
		31 July, 1980		
90	40.431	138.470	0	0
120	302.597	320.420	23	23
150	364.668	364.668	79	79
180	372.006	379.434	490	490
240	379.434	379.434	2300	2300
300	379.443	379.434	1700	1700
		1 August, 1980		
90	67.274	186.70	8	49
120	277.254	277.254	79	79
150	350.260	372.006	790	790
180	372.006	372.006	4900	4900
240	372.006	372.006	2300	2300
300	372.006	372.006	1100	1100
		2 August, 1980		
90	34.984	127.670	8	13
120	289.756	314.710	17	33
150	343.188	343.188	1300	1300
180	394.560	394.560	1300	1300
240	394.560	394.560	1700	1700
300	394.560	394.560	1100	1100

Table 21. Data for 12.19 m (40 ft) source distance from the concrete runoff surface.

Time (sec)	Flow (cm^3/sec)	Corrected Flow (cm^3/sec)	Fecal Coliform (MPN/100 ml)	Total Coliform (MPN/100 ml)
		4 August, 1980		
90	31.617	120.740	0	5
120	283.463	313.770	2	2
150	357.419	357.419	490	490
180	357.419	357.419	3300	3300
240	357.419	357.419	4600	4600
300	357.419	357.419	4900	4900
		5 August, 1980		
90	38.561	134.810	0	2
120	283.463	299.510	5	5
150	372.006	379.140	4900	4900
180	386.952	402.260	22000	54000
240	402.260	402.260	4600	4600
300	402.260	402.260	22000	22000
		6 August, 1980		
90	46.368	149.750	0	2
120	277.254	277.250	23	23
150	357.419	372.006	790	790
180	372.006	372.006	2300	2300
240	372.006	372.006	3300	3300
300	372.006	372.006	4600	4600
		7 August, 1980		
90	69.896	191.100	2	2
120	289.756	289.756	33	33
150	329.310	329.310	7900	7900
180	336.205	336.205	13000	13000
240	343.188	343.188	2300	2300
300	357.419	357.419	7900	7900
		8 August, 1980		
90	86.935	218.84	0	5
120	302.597	302.597	33	33
150	386.952	386.952	110	110
180	386.952	386.952	460	460
240	386.952	386.952	7000	7000
300	386.952	386.952	2300	2300

Table 22. Data for 15.24 m (50 ft) source distance from the concrete runoff surface.

Time (sec)	Flow (cm^3/sec)	Corrected Flow (cm^3/sec)	Fecal Coliform (MPN/100 ml)	Total Coliform (MPN/100 ml)
		6 July, 1980		
90	135.938	292.800	0	5
120	501.888	501.888	2	2
150	615.335	615.335	330	330
180	635.632	635.632	3300	3300
240	656.334	656.334	24000	24000
300	666.838	666.838	13000	13000
		8 July, 1980		
90	161.156	328.710	5	23
120	538.136	538.136	5	43
150	615.335	615.335	330	490
180	615.335	615.335	490	490
240	615.335	615.335	7000	7000
300	615.335	615.335	22000	22000
		15 July, 1980		
90	38.561	134.810	0	0
120	283.463	299.510	2	5
150	305.260	307.040	79	79
180	379.434	384.780	1100	1100
240	402.260	402.260	3500	3500
300	379.434	379.434	13000	13000
		10 August, 1980		
90	8.463	61.940	0	0
120	219.718	301.710	5	8
150	343.188	343.188	490	490
180	372.006	372.006	1300	1300
240	386.952	386.952	4900	4900
300	372.006	372.006	7900	7900
		9 August, 1980		
90	4.264	45.260	0	0
120	289.756	289.756	5	5
150	357.419	357.419	80	80
180	357.419	357.419	330	330
240	357.419	357.419	790	790
300	357.419	357.419	3300	3300

Table 23. Data for 22.86 m (75 ft) source distance from the concrete runoff surface.

Time (sec)	Flow (cm^3/sec)	Corrected Flow (cm^3/sec)	Fecal Coliform (MPN/100 ml)	Total Coliform (MPN/100 ml)
		11 July, 1980		
90	46.368	149.75	0	220
120	467.176	513.520	0	2
150	635.632	635.632	2	23
180	635.632	635.632	33	49
240	635.632	635.632	490	490
300	635.632	635.632	1700	1700
		13 July, 1980		
90	50.607	157.560	2	49
120	253.255	253.255	0	49
150	372.006	372.006	23	23
180	372.006	379.434	27	79
240	379.434	386.952	790	790
300	386.952	386.952	1300	1300
		18 July, 1980		
90	42.355	142.170	2	8
120	277.254	284.380	0	0
150	336.205	345.120	0	0
180	350.260	350.260	49	79
240	350.260	350.260	2200	2200
300	357.419	357.419	3300	3300
		11 August, 1980		
90	28.456	114.010	2	11
120	277.254	312.900	0	2
150	315.780	315.780	0	2
180	315.780	315.780	210	210
240	315.780	315.780	330	330
300	315.780	315.780	2800	14000
		12 August, 1980		
90	15.575	83.310	5	5
120	184.211	225.210	11	11
150	350.260	359.170	33	33
180	372.006	372.006	23	23
240	372.006	372.006	330	330
300	372.006	372.006	790	790

Table 24. Data for 27.43 m (90 ft) source distance from the concrete runoff surface.

Time (sec)	Flow (cm^3/sec)	Corrected Flow (cm^3/sec)	Fecal Coliform (MPN/100 ml)	Total Coliform (MPN/100 ml)
		12 July, 1980		
90	55.074	165.590	49	79
120	315.780	315.780	22	49
150	417.933	417.933	49	49
180	442.135	442.135	2300	2300
240	467.176	467.176	24000	24000
300	450.388	450.388	24000	24000
		14 July, 1980		
90	20.163	95.030	0	4
120	219.718	258.930	0	0
150	315.780	315.780	0	0
180	343.188	343.188	70	110
240	357.419	357.419	790	790
300	357.419	357.419	700	700
		17 July, 1980		
90	96.321	233.570	0	17
120	410.051	410.051	0	2
150	585.642	585.642	2	2
180	585.642	585.642	70	70
240	585.642	585.642	4600	4600
300	585.642	585.642	1700	1700
		13 August, 1980		
90	12.597	74.980	0	0
120	219.718	283.890	8	8
150	329.310	329.310	2	2
180	350.260	350.260	130	130
240	350.260	350.260	33	33
300	350.260	350.260	33	33
		14 August, 1980		
90	31.617	120.74	49	49
120	236.122	252.160	11	11
150	364.668	372.006	13	13
180	372.006	372.006	330	330
240	372.006	372.006	490	490
300	372.006	372.006	330	330

Table 25. Data for 30.48 m (100 ft) source distance from the concrete runoff surface.

Time (sec)	Flow (cm^3/sec)	Corrected Flow (cm^3/sec)	Fecal Coliform (MPN/100 ml)	Total Coliform (MPN/100 ml)
		19 July, 1980		
90	31.617	120.740	0	0
120	289.756	321.840	2	5
150	329.310	329.310	0	0
180	329.310	329.310	0	0
240	322.501	322.501	330	330
300	329.310	329.310	1700	1700
		20 July, 1980		
90	22.734	101.160	5	5
120	230.574	266.220	0	0
150	320.597	302.597	0	0
180	329.310	329.310	0	0
240	350.260	350.260	130	130
300	289.756	289.756	130	130
		15 August, 1980		
90	4.759	47.540	0	0
120	219.718	323.100	0	0
150	343.188	343.188	0	0
180	343.188	343.188	8	8
240	350.260	350.260	230	230
300	350.260	350.260	330	330
		16 August, 1980		
90	31.617	120.740	13	79
120	253.255	274.640	2	8
150	364.668	364.668	11	17
180	364.668	364.668	17	79
240	364.668	364.668	70	70
300	364.668	364.668	490	490
		17 August, 1980		
90	31.617	120.740	0	33
120	241.752	259.580	0	8
150	343.188	353.880	0	2
180	372.006	372.006	0	0
240	343.188	343.188	170	170
300	343.188	343.188	460	460

Table 26. Data for control run of 19 June 1981 on soil surface with rainfall excess predicted by the Green and Ampt infiltration equation.

Time (min)	Rainfall Excess (cm/min)	Flow (cm^3/sec)	Corrected Flow (cm^3/sec)	Fecal Coliform (MPN/100 ml)	Total Coliform (MPN/100 ml)
0					
	0.013				
5		39.696	49.321	4900	4900
	0.035				
10		248.865	249.222	46000	46000
	0.039				
15		382.952	382.952	7900	49000
	0.041				
20		382.952	382.952	33000	49000

Table 27. Data for 1.52 m (5 ft) source distance run of 22 June 1981 on soil surface with rainfall excess predicted by the Green and Ampt infiltration equation.

Time (min)	Rainfall Excess (cm/min)	Flow (cm^3/sec)	Corrected Flow (cm^3/sec)	Fecal Coliform (MPN/100 ml)	Total Coliform (MPN/100 ml)
0					
5					
	0.009				
10		32.547	41.959	170000	170000
	0.015				
15		131.879	135.800	79000	79000
	0.017				
20		156.944	156.944	170000	170000

Table 28. Data for 3.04 m (10 ft) source distance run of 23 June 1981 on soil surface with rainfall excess predicted by the Green and Ampt infiltration equation.

Time (min)	Rainfall Excess (cm/sec)	Flow (cm^3/sec)	Corrected Flow (cm^3/sec)	Fecal Coliform (MPN/100 ml)	Total Coliform (MPN/100 ml)
0					
5					
	0.009				
10		13.945	20.362	3300	4900
	0.015				
15		131.879	134.375	49000	49000
	0.017				
20		156.944	156.944	23000	33000

Table 29. Data for 15.24 m (50 ft) source distance run of 24 June 1981 on soil surface with rainfall excess predicted by the Green and Ampt infiltration equation.

Time (min)	Rainfall Excess (cm/sec)	Flow (cm^3/sec)	Corrected Flow (cm^3/sec)	Fecal Coliform (MPN/100 ml)	Total Coliform (MPN/100 ml)
0					
5					
	0.027				
10		66.618	78.382	22000	22000
	0.033				
15		272.889	280.375	23000	23000
	0.036				
20		311.500	311.500	7900	7900

Table 30. Data for control run of 26 June 1981 on soil surface with rainfall excess predicted by the Green and Ampt infiltration equation.

Time (min)	Rainfall Excess (cm/sec)	Flow (cm^3/sec)	Corrected Flow (cm^3/sec)	Fecal Coliform (MPN/100 ml)	Total Coliform (MPN/100 ml
0					
5					
	0.009				
10		43.601	53.583	3300	3300
	0.015				
15		116.630	123.403	4900	4900
	0.017				
20		156.944	156.944	6300	6300