STORMWATER DETENTION PONDS: AN EVALUATION USING FREQUENCY DISTRIBUTIONS FOR DETENTION TIMES

BY

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THESIS

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ABSTRACT

Detention ponds are used to manage both the peak discharges and pollutant loads in runoff waters. To assess the effectiveness of detention ponds, time consuming and costly field sampling can be done. However, an alternative means of assessment is proposed. It is detention time, which may be a prediction method to assist in the determination of the pollution removal efficiency of a detention pond. Detention time is commonly calculated by dividing the pond volume by the outflow rate. Using detention time frequency distributions, comparisons are made for two existing pond design criteria and four proposed pond design criteria. These detention time frequency distributions are compared with frequency distributions for time between outflow events. The time between outflow events is the time from the start of pond outflow to the start of outflow from the next rainfall event.

The object is to determine a detention pond design for which the frequency distribution for detention time is about equal to the frequency distribution for time between outflow events. This study was limited to two sites located in the central Florida area, namely a residential and a commercial site. Using Department of Environmental Regulation design criteria, detention time frequency distribution approximates the time between storms.

DEDICATION

This thesis is dedicated to my wife, Colene, and to my thesis chairman and friend, Dr. Martin P. Wanielista.

ACKNOWLEDGEMENTS

I would like to acknowledge all who have played a role in my accomplishing this thesis. More specifically, I would like to acknowledge Mr. Bob Laumer, who helped me in the field in both the preparation and the collection of data. I would like to thank Dr. Shiou S. Kuo and Dr. Yousef A. Yousef, both of who were on my committee and played a role in both my graduate and undergraduate years here at the University of Central Florida. They worked patiently with me. I would like to especially thank, Dr. Marty Wanielista for his unmeasurable support and motivation. Undoubtedly, without his patience and sincere concern for both myself and the research, this thesis would not have become a reality. Thank you, Dr. Wanielista. And finally, a special thanks to Mrs. Joyce Lee, Mrs. Peggy Dall, and Ms. Carol Ann Pohl in the Civil Engineering and Environmental Sciences Department who always work hard and always offer a helpful hand and timely reminders.

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CHAPTER 1

INTRODUCTION

Detention ponds are used to manage both the peak discharges and pollutant loads in runoff waters. A detention pond, as defined by the Florida Department of Environmental Regulation, is a structure used for "the collection and temporary storage of stormwater in such a manner as to provide for treatment through physical, chemical, or biological processes with subsequent gradual release of the stormwater" (Florida Administrative Code 17-25.020 1982). To assess the effectiveness of detention ponds, time consuming and costly field sampling can be done. However, an alternative means of assessment is proposed. It is detention time, which may be a prediction method to assist in the determination of the pollution removal efficiency of a detention pond. Detention time is commonly calculated by dividing the pond volume by the outflow rate. For variable inflow and outflow rates, another formula for detention time has been proposed and is based on both inflow and outflow rates (Nix 1985).

Objectives

Using two existing and operating detention ponds, the purpose of this work is to describe with frequency distributions, the variability in detention time for variable inflow rates, outflow rates, and pond volumes. Using detention time frequency distributions, comparisons are made for two existing pond design criteria and four proposed pond design criteria. These detention time frequency distributions are compared with frequency distributions for time between pond outflow events. The time between outflow events is the time from the start of pond outflow to the start of outflow from the next rainfall event. A rainfall event is defined by a period of time during which rainfall produces runoff and there is no rainfall for at least 4 hours before and after the rainfall period.

The object is to determine a detention pond design for which the frequency distribution for detention time is about equal to the frequency distribution for time between outflow events. The time between outflow events is the maximum holding time for a specific mixture of water, sediment, and chemicals. The pond mixture will change when another inflow event occurs.

Limitations

This study was limited to two sites located in the central Florida area, namely a residential and a commercial site. The commercial site was designed using bank exfiltration as the primary outflow mechanism. Also, there exists a surface discharge over a sharp crested rectangular weir, while the primary means of discharge at the residential site was by surface flows using a combination of orifices and weirs. The focus of this study was on the hydrologic and hydraulic characteristics of the watersheds and ponds. Water quality data were collected, but not reported here.

For the calculation of detention time, the total pond volume was assumed to be reactive with inflow water or there was no short circuiting in the pond. Continuous monitoring of pond levels over a year were not possible for each site, thus pond levels after each runoff event were estimated and at the residential site were assumed to be at the control elevation when the estimated level was below the control.

CHAPTER 2

LITERATURE REVIEW

Rainfall-Runoff Process

The relationship between rainfall and runoff is predictable given accurate information regarding a specific watershed and rainfall. As rainfall occurs at a rate of intensity (i) for a given duration (D) a specific amount of rainfall excess can be calculated (Wanielista 1989). As this rainfall excess is routed to a detention pond over time, it becomes an inflow rate to the pond. The inflow rate (Q_I) to an existing wet detention facility can be monitored and recorded. When the volume of inflow is significant enough to increase pond depth, an outflow rate (Q_O) results. The outflow rate also can be recorded with time. At any time interval using inflow and outflow data, an instantaneous pond volume (V) and depth can be calculated. For existing ponds, depth and volume can be recorded directly. A schematic representation of inflow, outflow, and volume is shown in Figure 1. When the rate of inflow is greater than the rate of outflow, the pond volume does increase, and similarly when the rate of outflow exceeds the rate of inflow, the pond volume decreases. Using the data for inflow, outflow, and volume, a detention time (t_d) can be calculated instantaneously and plotted when outflow exists. However, since the detention time represents the time water would remain in the pond,

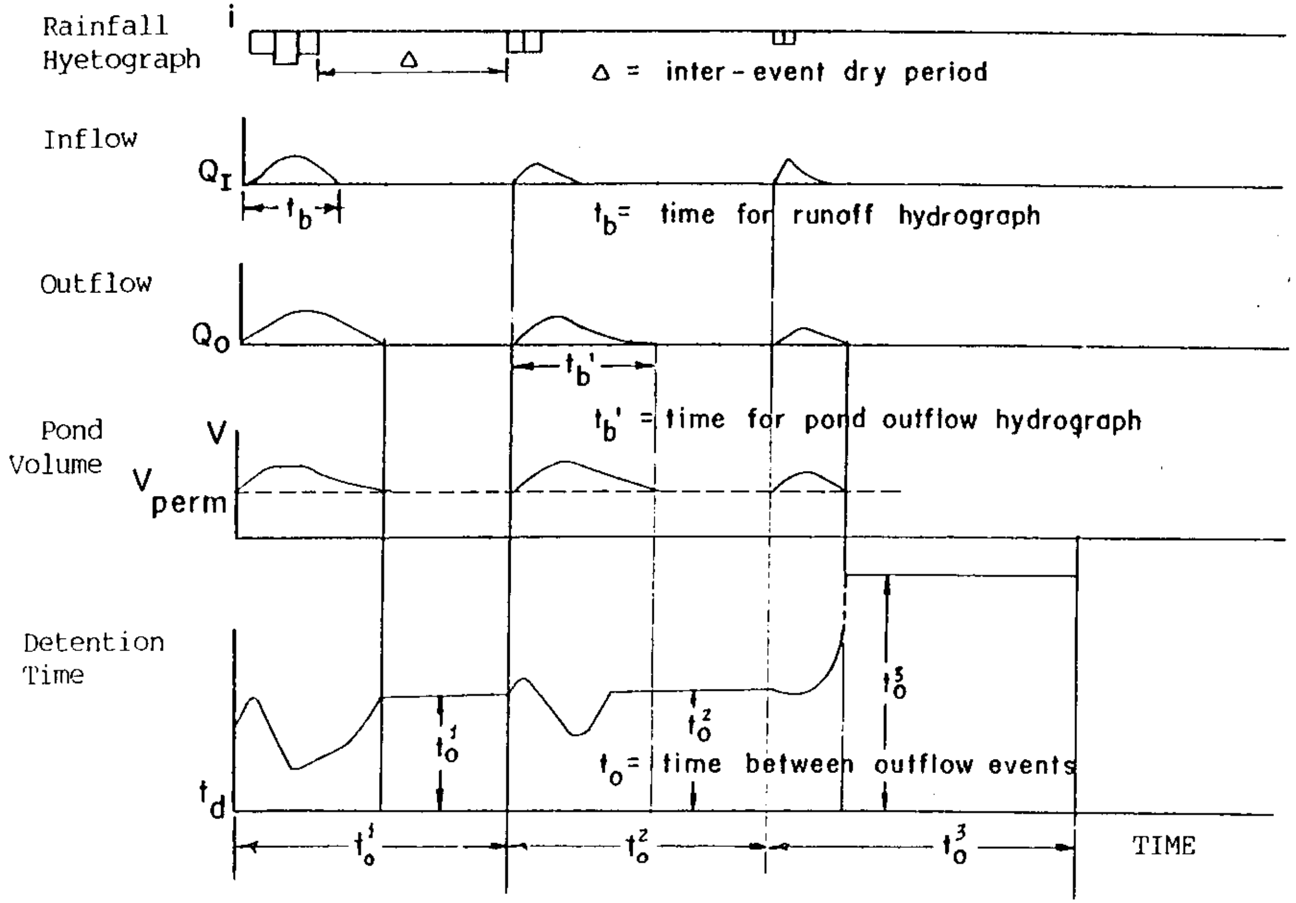

Figure 1. Schematic of Rainfall Runoff Processes with Wet Detention Pond Volume and Detention Time.

actual detention time must also include the time period between outflow events.

Significant Terminology

Detention time is a term often used interchangeably with the Florida Department of Environmental Regulation (FDER) term residence time, which is defined as the average length of time that a parcel of stormwater runoff resides in a detention facility (Camp Dresser & McKee 1985). However, detention time can change over time, and thus is a more exact term.

Other significant terms as shown in Figure 1 are:

1. The interevent dry period (Δ) or the period of time from the end of rainfall from one independent rainfall event to the beginning of the next independent rainfall event. Generally four hours is the minimum time for storms to be considered independent (Hvitved-Jacobsen, Yousef 1987). A rainfall of 0.04 inches or more was necessary to produce a measurable runoff.
2. The time between outflow events (t_o) is the period of time from the beginninng of outflow from one rainfall-runoff event to the beginning of outflow of the next independent rainfall-runoff event. It is a measure of the maximum detention time for the most recent mixture of runoff and pond water.

Models

Mathematical models are frequently useful to estimate detention time for detention ponds. Various models have been proposed. The continuous stirred tank reactor (CSTR) is one such model in which the contents are rapidly and continuously mixed. There is no difference in concentration of any species anywhere in the tank (Wanielista, Yousef, Taylor, and Cooper 1984). The plug flow reactor (PFR) is another model where flow is assumed to be one dimensional, velocity is constant across the pond and dispersion is assumed to be negligible (Wanielista, Yousef, Taylor, and Cooper 1984).

Detention ponds are categorized as one of three types: (1) plug flow, (2) completely mixed or (3) intermediately mixed. The plug flow pond queues flow such that flow parcels leave the pond in the same order they entered. In completely mixed ponds, flow parcels are immediately and uniformly dispersed throughout the pond. Any pond demonstrating a level of mixing between these two extremes is classified as intermediately mixed (Nix 1985). Frequency distributions are used to illustrate comparisons among the three models as shown in Figure 2.

Martin (1988) conducted detention time studies on an 8600 square foot pond with 53,040 ft^3 of dead storage. A frequency distribution was determined for detention time and it best approximated the intermediately mixed situation of Figure 2. The mean value of detention time always exceeded the median value for each runoff event indicating a right skewed distribution for detention time.

Levenspiel (1962) introduced the mathematical concepts of intermediately mixed systems which he called arbitrary flow systems. Levenspiel used internal age and exit age probability distributions to describe the detention time of a particular system. The internal age distribution is a probability distribution for fluid elements within a pond. It is measured from the time particles enter the system, whereas the exit age is a probability distribution for elements leaving the pond. It is the measure of detention times for all fluid

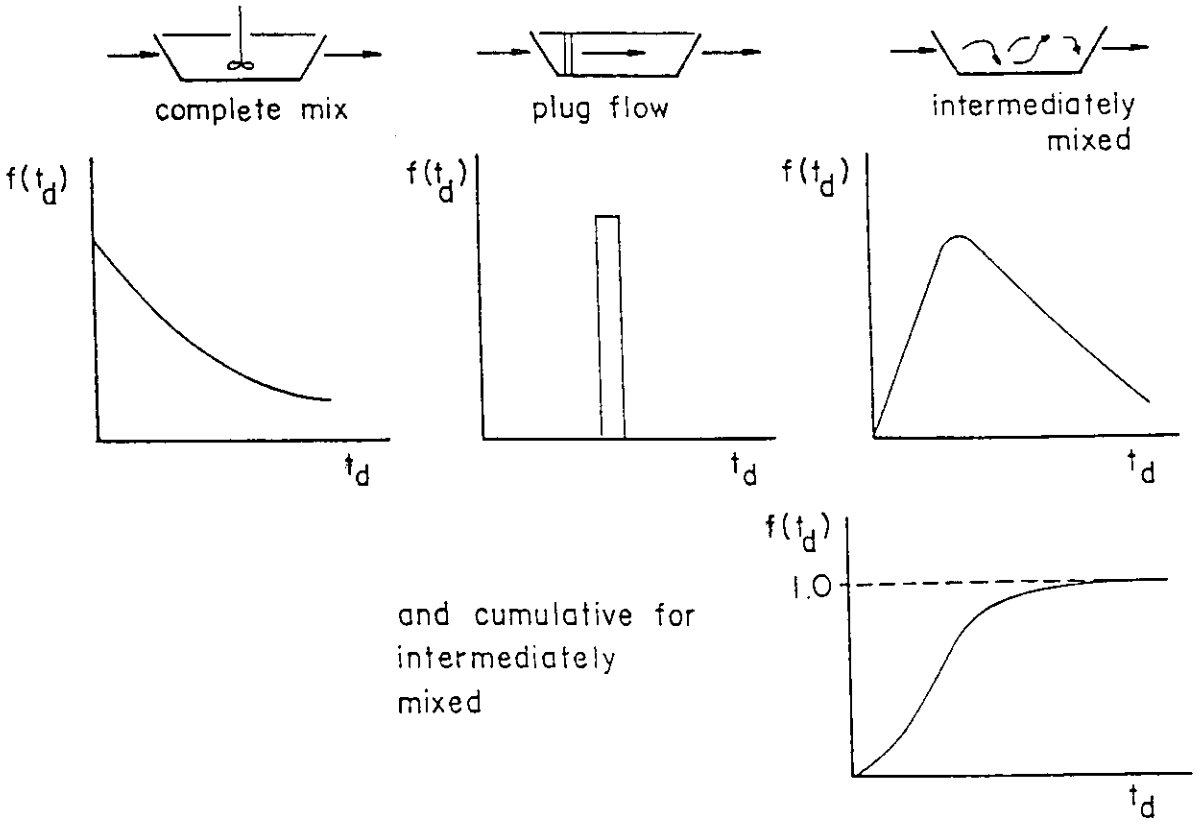

Figure 2. Frequency Distribution for Three Flow Models.

particles discharging from the pond (Levenspiel 1962). The distribution for exit age (detention time) approximates that of the intermediately mixed distribution of Figure 2. From the past work of Martin (1988), Nix (1985), and Levenspiel (1962), a stormwater detention pond will most likely function and produce a frequency distribution similar to that characteristic of an intermediately mixed pond.

Formulas for Detention Time

Detention times are calculated to determine the length of time a water molecule remains in a detention facility. Detention time as calculated for stormwater detention ponds is a measure which varies with time because inflow rates, outflow rates, and pond volume vary with time. No one number completely describes the detention time. However, considering a steady state outflow rate and a constant volume without reaction and without density change of a single fluid through a pond, detention time is defined as (Levenspiel 1962):

$$t_d = \frac{V}{Q_o} \tag{1}$$

where: V = Average volume of pond during a time period (cf)

Q_o = Average outflow rate during a time period (cfs)

t_d = Detention Time (s)

Since flow rate and volume change with time, the formula for variable detention time is written:

$$t_d(t) = \frac{V(t)}{Q_o(t)} \tag{2}$$

where: $V(t)$ = Volume of Pond with Time (cf)

$Q_o(t)$ = Outflow Rate with Time (cfs)

Using equation (2), a frequency distribution for any interval of time can be developed to provide a representation of detention time as a hydrologic process.

Simulations performed by the Storm Water Management Model (SWMM) Storage/Treatment Block using the Puls method suggested that a better single estimate is obtained if the volume of the water in the pond is divided by the average of the inflow and the outflow rates (Nix 1985):

$$t_d = \frac{2V}{Q_I+Q_O} \tag{3}$$

where: Q_I = Inflow Rate (cfs)

Pond Inflow Rates

In a flow through system outflow rates may depend on inflow rates to a pond and thus it is important to estimate inflow rates. Several methods for hydrograph generation (inflow rates) exist. The most popular methods are the rational method, SCS, and Santa Barbara Urban Hydrograph method (SBUH). The rational method is used most frequently for calculating peak runoff for short time of concentration ($\leq$ 20 minutes) watersheds (Wanielista, in press). The peak is determined by multiplying the runoff coefficient by both the intensity and area of the contributing watershed. The peak usually occurs at the end of time of concentration (Wanielista, in press). However, the rational formula is not widely used for predicting the shape of a hydrograph.

For the SCS method developed by the Department of Agriculture (1986), the peak runoff is determined by the following equation:

$$Q_p = \frac{KAR}{t_p} \tag{4}$$

where: A = Area (mi^2)

R = Total Runoff (in)

t_p = Time to Peak (hr)

Q_p = Peak Runoff (cfs)

K = Attenuation Factor (cfs/mi^2-in/hr)

Where the time to peak is calculated using:

$$t_p = \frac{D}{2} + 0.6\ t_c \tag{5}$$

where: D = Duration (hr)

t_c = Time of Concentration (hr)

The attenuation factor determines the shape of the hydrograph. The common shape is triangular with the recession limb time equal to 1.67 times the time to peak which produces a "K" factor of 484 (Wanielista, in press).

Using the SBUH method (Stubchaer 1975), the pond inflow hydrograph is obtained by routing the instantaneous hydrograph for each time period (usually 15 minutes) through an imaginary linear reservoir with a routing constant calculated from the time of concentration of the watershed (Wanielista, in press).

$$Q(t+1) = Q(t) + K_u[I(t+1) + I(t) - 2Q(t)] \tag{6}$$

where: $Q(t+1)$ = routed flow in time $(t+1)$, cfs

$Q(t)$ = routed flow in time (t), cfs

$I(t+1)$ = rainfall excess in time $(t+1)$, cfs

$I(t)$ = rainfall excess in time (t), cfs

$$K_u = \frac{\Delta t}{[2t_c + \Delta t]} \qquad (7)$$

and Δt = time interval, units consistent with t_c

Pond Outflow

The release rate of the pond water is an essential variable for the design of detention ponds. According to current DER criteria, detention ponds shall release 1/2 of the runoff from the design storm in at least 60 hours (Livingston 1988). To meet this requirement, an investigation of inflow, outflow, and pond volume is necessary.

The fundamental laws that govern and describe fluid flow are described by the Momentum and Continuity Equations (Wanielista, in press). There exist in common use, at least two hydrologic flood routing methods for routing an inflow hydrograph through storage in a reservoir, river, or stormwater detention pond. The inventory method, and the Muskingum formula are two common ways of flood routing using the continuity equation (Wanielista, in press). Both methods assume a relationship between the pond storage and an outflow hydrograph with the storage being dependent on previous outflow and inflows (Wanielista, in press). Figure 3 illustrates a typical inflow and outflow hydrograph relationship. Note that the volume of detention storage can be calculated from Figure 3 as the difference between inflow and outflow volumes. The mass balance or inventory equation for a pond must be thoroughly documented and calibrated if it is to be used to predict outflow rates. This is important when groundwater inflow into or from a detention pond is possible.

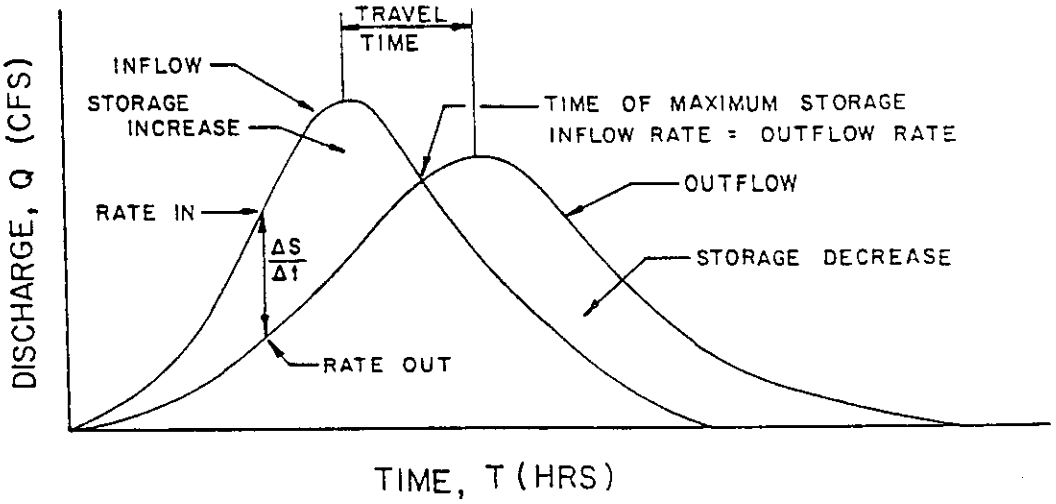

Figure 3. Inflow and Outflow Hydrographs.

Most of the commonly used methods of flood routing are based solely on the solution of the inventory equation which provides for conservation of mass (Wanielista, in press). The inventory equation as applied to reservoir routing states that the volume of inflow minus the volume of outflow over a given time interval is equal to the change in volume stored over that time interval. Therefore, an inventory can be written as follows:

$$S_2 - S_1 = \bar{I}(\Delta t) - \bar{O}(\Delta t) = S = [(I_1+I_2)/2](\Delta t) - [(O_1+O_2)/2](\Delta t)$$

where: $\bar{I}$ = The average inflow during the time step, t, L^3/T

$\bar{O}$ = The average outflow during the time step, t, L^3/T

S = The volume stored during time step $L^3, \Delta t$

The outflow volumes of the inventory equation are calculated knowing mathematical relationships for orifices, weirs, and bank infiltration. All types of outflow devices can be characterized by a mathematical equation (i.e., exponential, power or linear equations).

To effectively model the outflow, an accurate storage/outflow rate relationship must be identified.

The Muskingum method for flood routing was developed for the Muskingum Conservency District flood control study in the 1930's. The Muskingum method is used primarily for streams and not ponds or reservoirs.

Sedimentation

Sedimentation is the process by which sediment and some particulate forms of nutrients, metals, and attached bacteria settle to the bottom of ponds. Sedimentation efficiencies are commonly related to the settling velocities of each particle and hence detention time. Tests conducted by the Nationwide Urban Runoff Program (NURP) indicated the following findings. (1) There is a wide range of particle sizes, and hence settling velocities in any individual urban runoff sample. (2) There is substantial storm-to-storm variability in median (or other percentiles of) settling velocity, at a specific site. Among storm events, the median range reported was approximately one order of magnitude. (3) There were significant differences between the average settling velocities among sites (Driscoll 1983).

Several recent studies have suggested that two of the most important design criteria for a detention pond is the storage capacity of the permanent pool and detention time during storm runoff events (Camp Dresser & McKee 1985). Monitoring data indicates that a relatively large permanent pool will typically achieve greater than average removal rates than a relatively small permanent pool (Camp

Dresser & McKee 1985). Further, evaluations of variations in pollutant removal rates from storm-to-storm indicate higher efficiencies when storm runoff volumes are relatively smaller than the permanent pool and vice versa (Camp Dresser & McKee 1985). Studies done by the Metropolitan Washington Council of Governments, Northern Virginia Planning District Commission, and Occoquan Watershed Monitoring Laboratory, showed that as average detention time approached 50 hours the percent sediment removed reached levels between 70 and 90 percent using experimental settling column studies (Camp Dresser & McKee 1985).

State Criteria

The initial State of Florida regulation for stormwater control was promulgated in Chapter 17-4.248 Florida Administrative Code Florida Stormwater Management Regulations. The focus of the regulation was on new stormwater discharges which had or were expected to have a "significant impact on water quality" (Camp Dresser & McKee 1985). The regulation did not provide specific performance standards or guidelines for applicants to use in the design of stormwater management facilities. Chapter 17-25 Florida Administrative Code, a revision of Chapter 17-4.248 Florida Adminstrative Code, came into effect February 1, 1982. In addition to providing permitting requirements, the regulation provided performance standards to be used in the design of stormwater management facilities (Camp Dresser & McKee 1985). The 1982 version of the "Basis of Review" designates the following stormwater management practices as acceptable best management practices for water quality management: (1) detention ponds with filtration; (2)

detention ponds without filtration; and (3) retention facilities (Camp Dresser & McKee 1985).

Some current design standards for detention ponds require (Livingston 1988):

1. 1" of runoff storage above the permanent pool;
2. No more than 1/2 of this volume to be discharged in 60 hours following an event;
3. The permanent pool must provide an average residence time of at least 14 days. This is approximately equivalent to the runoff volume of 2 inches times the impervious area and 1/2 inch times the pervious area;
4. Mean depth of 3-10 feet for the permanent pool;
5. Thirty percent of the pond area has established littoral shelf.

These newer design standards on permanent pool volume will be compared to older existing design standards to determine changes in detention time frequency distributions.

CHAPTER 3

SITE DESCRIPTIONS AND FIELD PROCEDURES

Residential Site (Greenview Park)

The residential site is located in East Orlando, approximately five miles north of the Orlando International Airport. The pond serves as a receiving water body for the runoff from the Greenview Park residential community, a watershed approximately 19.2 acres in size. The watershed is made up of 1/4 acre lots with single family homes.

The watershed time of concentration was estimated at 25 minutes, the percent impervious area was initially estimated at 44% with the percent directly connected impervious area estimated at 70% of the impervious area. From a field inspection of the watershed, all the homes had roofs with drain pipes primarily draining to the sodded pervious lawns and only some of the roofs drained onto directly connected impervious areas. It was impossible to exactly estimate the percent directly connected areas. Thus, field collected runoff and rainfall data would be used to calibrate a rainfall-runoff model and determine an estimate for percent directly connected areas. The watershed has minimal obstructions in the gutters leading to the inlets which lead to the underground drainage system. The roadways and gutters are periodically (once every 2 weeks) cleaned by street sweepers. The pond receives all runoff from the residential watershed as verified by observation during a rainfall-runoff event and by

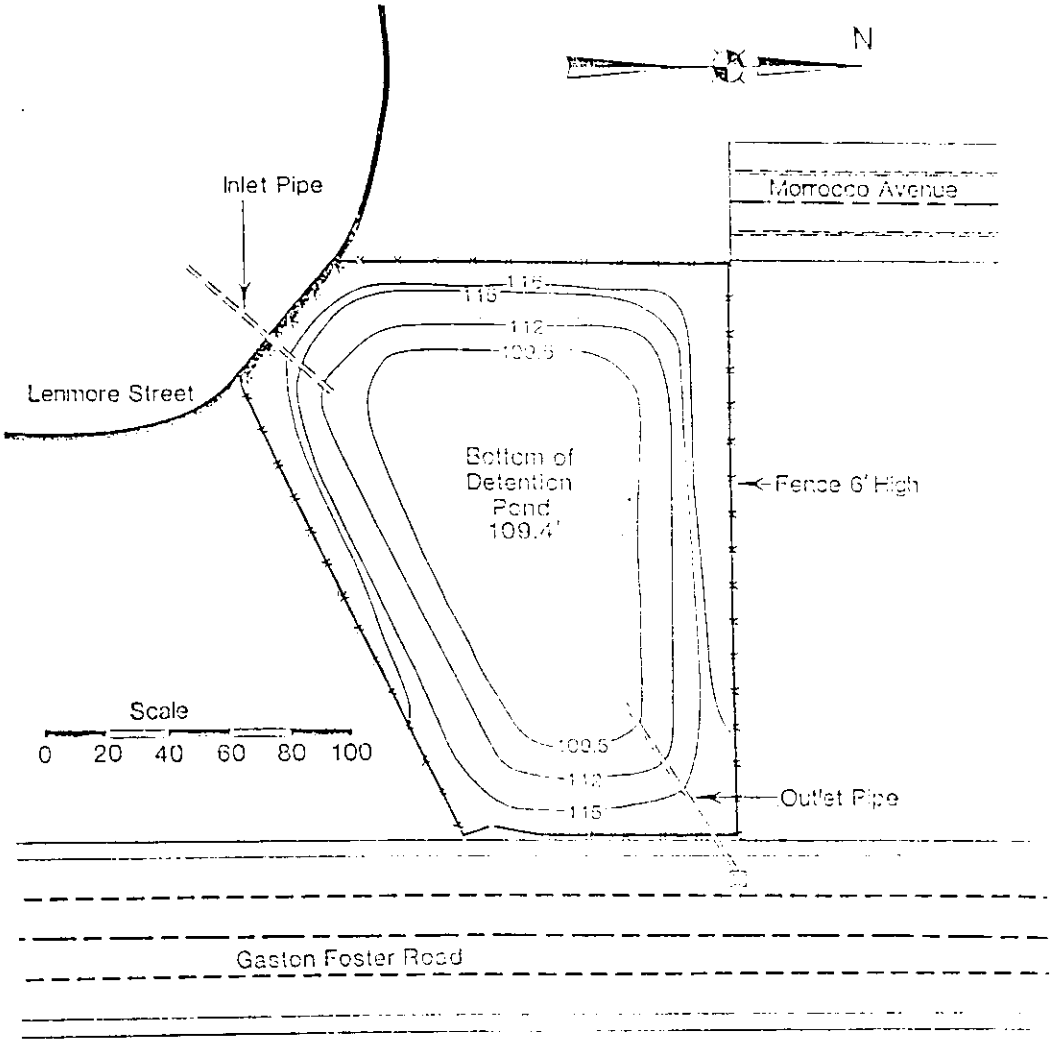

Figure 4. Residential Site - Pond Depth Map.

topographical maps. The pond geometry is shown in Figure 4. It has a dead storage of 24,064 cubic feet at a depth of about 3 feet (approximately 0.35 inches over the watershed).

Commercial Site (Research Park)

The commercial site is located 13 miles east of Orlando. The pond serves as a receiving water body for the runoff from a 0.95 acre commercial/manufacturing parcel. The watershed is composed of a parking area (65% of the total land) and one building with minimal (<0.01 acre) green areas. All impervious areas are directly connected.

The watershed time of concentration of 10 minutes was estimated using field experimentation. A fire hydrant located at the upper end of the watershed was opened and the time to drain into the detention pond was recorded. The percent impervious area is about 99% with the percent directly connected impervious area of 100%. These were estimated from a field survey. The watershed has one grated inlet with minimal obstructions, which lead to the underground drainage system and into a pond. The commercial area pond receives all runoff from the watershed as verified by topographical survey.

The pond depth contour map is shown in Figure 5. The dead storage is 1,671 cubic feet (approximately 1/2 inch over watershed).

Flow Measurements

The flow into the residential pond was measured using an ISCO velocity meter. The flow enters the pond by way of a 30-inch concrete pipe, which is completely submerged a large percentage of the time.

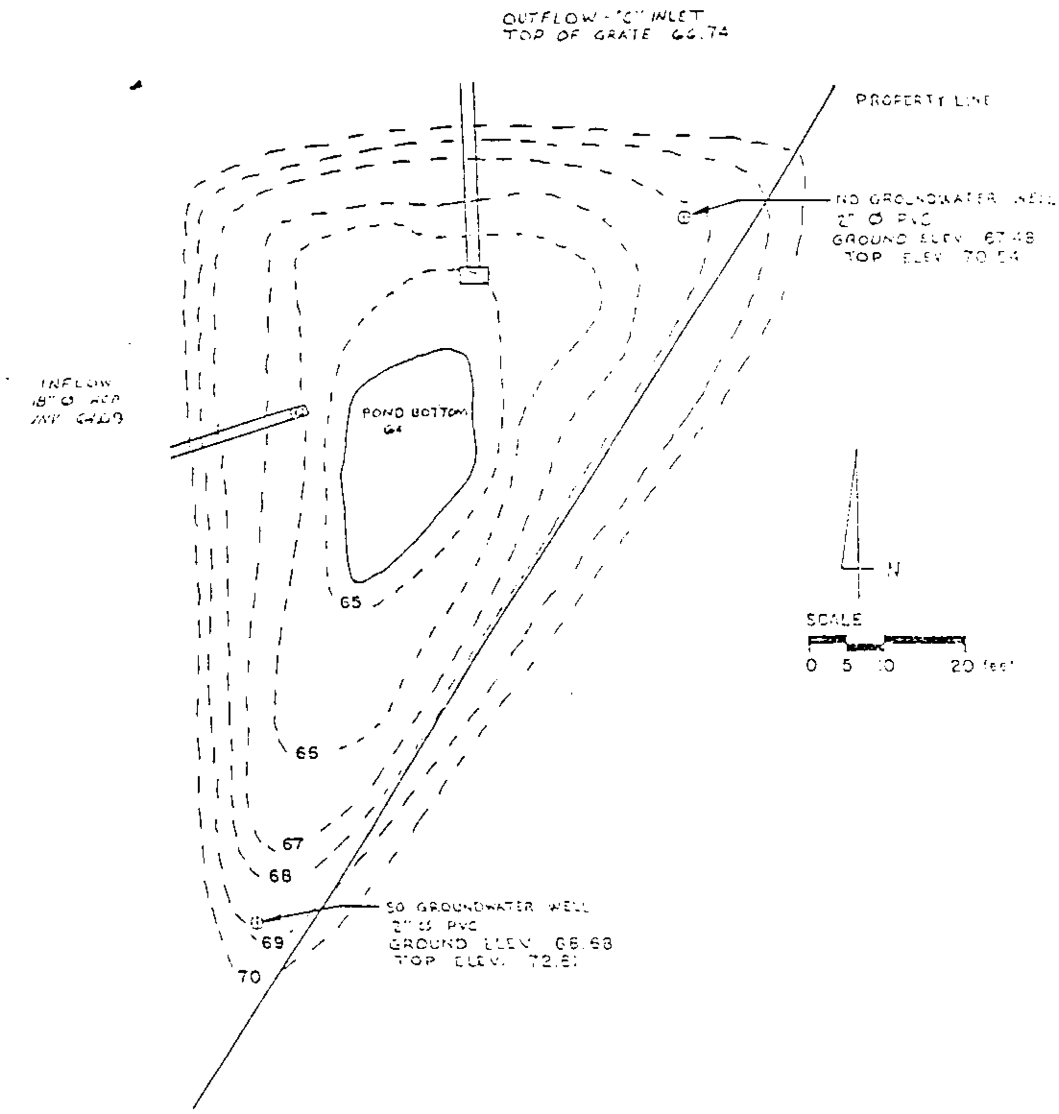

Figure 5. Commercial Site - Pond Depth Map.

Velocities were measured during the course of a runoff event, and pond depth was constantly recorded which would reflect an estimation of the pond volume. The measured inflow velocities over time and estimated pond volumes provide two of the variables for a pond inventory equation. The rainfall was measured on site using two rain gauges and verified using data received from the Orlando International Airport. The rainfall combined with the watershed data provided an expected volume of flow and was used to verify the rainfall excess calculations.

The flow out of the residential pond was measured using weirs, the "as-built" structure was a riser pipe with a 3 1/2 inch orifice, which provided a bleed down. The outflow structure was modified and measurement was done using a 90^{o} V-notch weir. The height of the water over the weir was measured using an ISCO pressure inducer which was calibrated to measure the height of the water column over the outflow invert. Once the height over the weir is known, a flow rate can be calculated using calibrated equations. The weirs were calibrated using direct measurement at various water heights over the weir.

The flow out of the research park pond was measured by using the ISCO pressure inducer to measure the depth of the pond at the weir, which was broadcrested. Once the depth of the pond was known, the broadcrested weir equation was used to determine the outflow rate.

Rainfall was measured directly at the research park using one rain gauge. The values were compared to data provided by a precipitation station located at the University of Central Florida (one-half mile northeast of the pond).

Modification of "As-Built" Pond Geometry

The "as-built" pond geometry at the residential site provided permanent pool storage for 24,064 cubic feet. This did not meet the current DER (1988) design criteria and was modified to do so. The discharge structure was changed from a rectangular weir to 90° V-notch weir and the orifice was eliminated. The modifications provided an additional 18,520 cubic feet of permanent storage for a total of about 0.61 inches over the watershed. The modification increased the depth of the pond by approximately one foot.

Outflow Calibration

The outflow control devices of both the residential and commercial detention ponds were calibrated. Also, over time the depths of the pond were recorded to calculate the change in pond volume and the depth over the weir. The outflow water was collected in calibrated containers over a period of time to directly measure the flow rate. This was designated as experimental outflow rate data. The height versus flow was plotted "best fitting" equations were determined. The field collected data for both sites are shown in Table 1.

TABLE 1

EXPERIMENTAL OUTFLOW-RATE DATA VERSUS DEPTH OVER WEIR

COMMERCIAL SITE			RESIDENTIAL SITE					
			AS-BUILT			MODIFIED (90° WEIR)		
DEPTH RATE		FLOW RATE	DEPTH*		FLOW RATE	DEPTH		FLOW
(in)	(ft)	(cfs)	(in)	(ft)	(cfs)	(in)	(ft)	(cfs)
0.38	.031	0.08	1.25	0.14	0.12	1.20	0.10	0.008
1.13	.094	0.50	11.00	0.95	0.37	1.80	0.15	0.020
1.75	.146	1.47	13.00	1.20	2.10	2.25	0.19	0.038
						3.00	0.25	0.074
						4.00	0.33	0.167
						6.00	0.50	0.450

The "best fit" equations were determined using a least-squares bi-variate regression procedure and were:

Commercial Site $H \leq 0.12'$ $\quad Q = 3.0\ (L_1)H^{1.42}$ (8)

Commercial Site $H > 0.12'$

$$Q = 3.0\ (L_2)(H - 0.12)^{1.42} + 3.0\ (L_1)H^{1.42} \qquad (9)$$

Residential "As-Built" Orifice Plus Weir

$$Q = 0.332\ H^{0.5} + 19.8(H - 1.1)^{1.5} \qquad (10)$$

Residential "Modified" 90° V-notch

$$Q = 2.64\ (H)^{2.54} \qquad (11)$$

where: Q = Outflow Rate, cfs

L_1 = 5.33 feet, length of lower weir

L_2 = 4.00 feet, length of upper weir

H = Height above invert, feet

The outflow rate curves (discharge versus head) as "best-fit" lines are shown in relation to the field collected experimental flow rate data in Figures 6, 7, and 8. For the 90° V-notch weir of Figure 8, the theoretical equation was used for the simulation because the parameters of the experimental equation were not significantly different from the theoretical.

Soil Characteristics

Soil samples were obtained from the banks and pond bottom at both the commercial and residential sites. The samples were evaluated based on grain-size distributions and permeabilities. The procedures indicated by Liu and Evett (1984) were used to perform the sieve

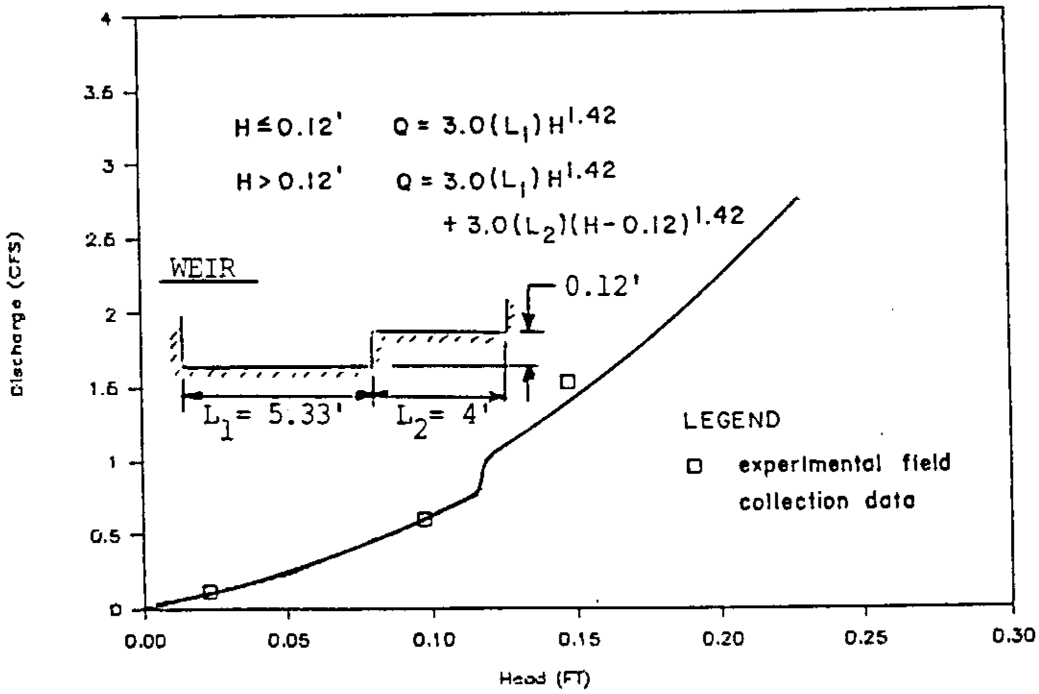

Figure 6. Outflow Calibration and "Best Fit" Curve Commercial Site.

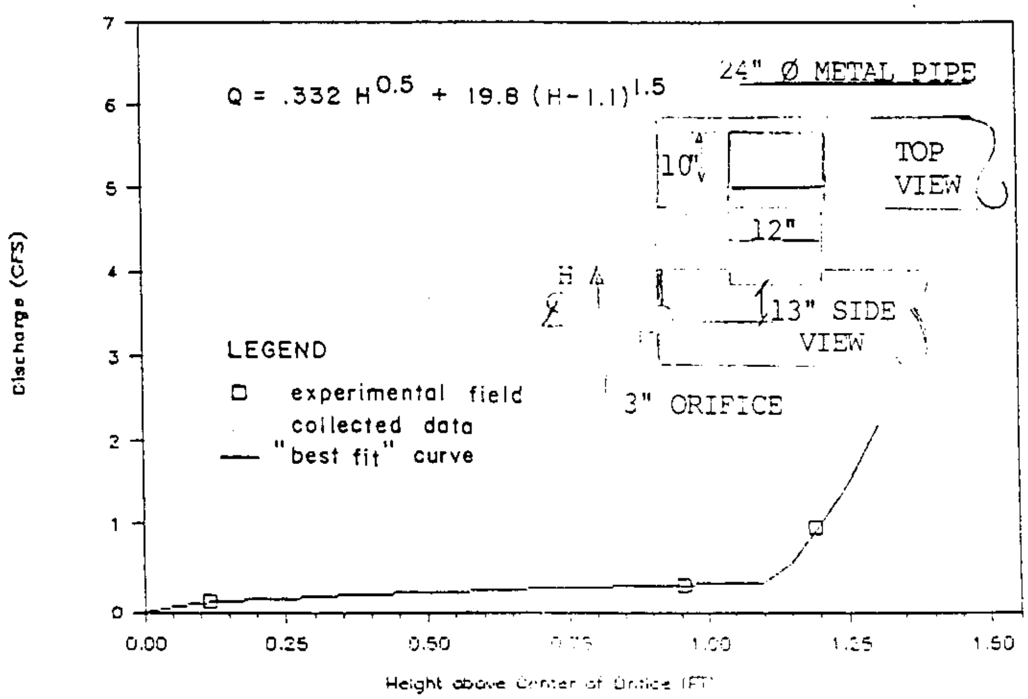

Figure 7. Outflow Calibration and "Best Fit" Curve Residential - "As-Built."

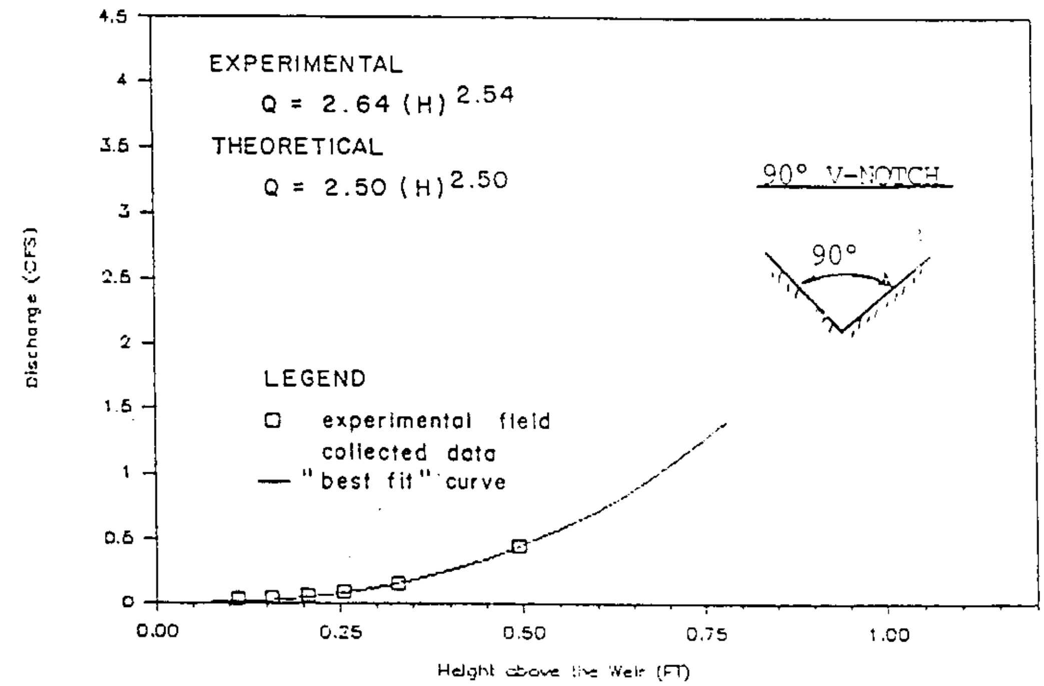

Figure 8. Outflow Calibration and "Best Fit" Curve Residential - Modified

analyses. Using the Asphalt Institute Soils Manual Classification, Table V-1 (Asphalt Institute 1978), the soils were classified as A-3, Fine Sand. The grain-size distribution curves are shown in Appendix A. From the grain size distribution coefficients of uniformity and gradation were calculated and are listed in Table 2.

The coefficient of uniformity, C_u, and coefficient of gradation, C_c, represent the characteristics of soil gradation which influences the ability for water to travel through the soil. The higher the values for C_u and the greater the deviation of C_c from 1.0, the higher the ability for water to flow through the soil. For sandy soils, a C_u of about 6 represents low permeability soils (Das 1985). The values obtained at both sites represent relatively permeable soils, which will permit the percolation of stormwaters from or into detention facilities.

TABLE 2

SOIL CLASSIFICATION

RESIDENTIAL SITE

LOCATION	DEPTH	C_u*	C_c*
Northeast	1 ft	1.72	0.94
Northeast	3 ft	1.72	0.84
Southwest	1 ft	1.93	0.95
Southwest	3 ft	2.00	0.84

where: $C_u = D_{60}/D_{10}$; $C_c = (D_{30})^2/(D_{60}xD_{10})$ and D_{xx} = diameter corresponding to xx% finer on particle size distribution.

COMMERCIAL SITE

LOCATION	DEPTH	C_u	C_c
Southwest	3 ft	1.60	0.90
Northeast	3 ft	1.87	0.86
Pond Bottom	.5 ft	1.75	0.75

The soils were classified as mostly Fine Sand in a Blanton series. This is not inconsistent with regional soil types. Blanton series soils are characterized by brownish-gray to dark gray with relatively high permeabilities. Approximately 11% of soils in Orange County Florida are Blanton Series (USDA 1960).

CHAPTER 4

MODEL FORMULATION AND CALIBRATION

Rainfall-Runoff Process

Because of equipment malfunctions with time and money constraints, it is impossible to constantly obtain over a number of years, direct measurement of rainfall, runoff, pond outflow, and pond volume. For the field site locations, there are approximately 124 independent rainfall events per year (Wanielista, in press). About 91 of these have rainfall volumes greater than 0.04 inches with a minimum inter-event dry period of 4 hours. For a 9 year period, using rainfall measurements at the Orlando Jetport, there were 819 rainfall events of volume greater than 0.04 inches separated by at least four hours of no rainfall (inter-event dry period). The average volume was 0.51 inches with a maximum of 5.04 inches. The average inter-event dry period was 90.6 hours with a maximum of 828 hours. Thus, to obtain frequency distributions for detention times over a yearly cycle, it is necessary to develop a mathematical model which can reproduce the hydrologic and hydraulic characteristics of the site specific watersheds and detention ponds.

Before any model can be used, it should reasonably simulate the actual field conditions. Thus, field collected rainfall, runoff, pond volume, and pond outflow rates were collected for eight storm events.

These data were used to calibrate a mathematical model for both the rainfall-runoff process and pond operation.

The runoff into the residential pond was estimated using a velocity meter and depth of flow in the inflow pipe. This runoff estimate was compared to the change of pond volume as measured by the pond depth gauge. A comparison of the volume of inflow calculated using inflow velocity and pond depth changes is shown in Table 3. These comparisons illustrate that the instruments give reasonable estimates for inflow rates. Both the depth gauge and the velocity meter were calibrated in a laboratory before field installation. During the field experiments, the orifice at the discharge pipe was blocked. The pond volume change during the rising limb of the influent hydrograph would lag behind the estimate of inflow using the velocity meter because of a travel time from the inlet to the pond depth measurement site.

TABLE 3

RESIDENTIAL POND
INFLOW VOLUME AND POND VOLUME COMPARISON

DATE	DURATION (min)	INFLOW VOLUME (cubic feet)	POND VOLUME (cubic feet)
11/19/87	30	2250	2080
11/19/87	60	4950	4468
12/5/87	60	2700	2301
12/5/87	60	6300	6390

An accurate measurement of rainfall volume for the watershed area was done using a rain gauge at both sites. Two gauges were used at the residential site and comparison of intensity and volume were made to compute averages for the 19.2 acre watershed. Once the volume data were collected for the experimental calibration events, a program was written on LOTUS 1-2-3 to estimate the runoff coefficient (C).

Similar experiments were done for both sites. However, on two occasions a fire hydrant located at the commercial site was used to supply flow, which could be directly read from a flow meter. This reading and the quantity into the pond were used in the determination of the infiltration rate through the pond bank at the commercial site. The infiltration rate also was calcualted from 8 other rainfall-pond storage events. The rate varied from a high of about 3 inches per hour at a pond depth equal to the invert of the surface discharge structure to zero at 2 feet below the invert. The average rate was about 0.25 inches per hour (see Appendix C for experimental data).

Determination of the runoff coefficient (C) was important for defining the watershed characteristics used to determine input hydrograph shapes. These C values could then be used to calibrate a hydrograph generation computer model. The computer model using field data was then used to estimate the pond input hydrograph.

Model Development

The model used for this analysis was a modified version of SMADA (Wanielista, in press). The modifications made on this program allow the calculation of detention times using the time variable equations (2.2 and 2.3), described earlier. These equations use outflow and the average of the inflow and outflow rates over a fixed time period. The model uses the SBUH method to generate the pond inflow hydrograph (runoff) and the volume-discharge relationship to describe the outflow hydrograph. Finally, using a pond mass balance, S = I-O, the pond volume can be determined. The calculation of the detention time is done for each 15 minute interval and averaged over a pond outflow event.

The model uses an analysis period equal to four times the storm duration. This period was chosen since hydrographs from both sites indicated that a rising limb equal to storm duration and a recession limb of the inflow hydrograph equal to three times duration would represent the expected flow rates. Also, pond outflow rates were minimal at the end of the analysis period. This also provided a similar basis to analyze each storm event rather than a variable period of time which would produce inconsistencies.

Inter-event Runoff Period - Detention Time Relationship

The period of time between outflow events was defined earlier as the inter-event runoff period, t_o. The interevent runoff time is also defined as the interval of time between the start of outflow from event (i) to the start of outflow from the next event (j). This period of time was used to establish a maximum detention time in a wet detention facility. Flow particles remaining in the detention pond after event (i) are affected directly by the beginning of flow from event (j). When the pond outflow ceases at the end of event (i) sedimentation and other processes are occurring on a specific mix of pollutants until the next event (j) occurs. Detention time is calculated as long as outflow occurs. When outflow equals zero, the detention time would equal the time to the next outflow event.

The inter-event runoff period can be calculated using the inter-event dry period and adding the storm duration. The average duration for rainfall in the study area is 4.3 hours. The maximum duration using 30 years of record was 54 hours. A frequency distribution for inter-event dry period would be different from the frequency distribution for detention time (inter-event runoff) by the duration of a storm event.

In modeling the commercial and residential sites, it was necessary to limit the calculated detention time so it did not exceed the inter-event runoff time. By inspection of equation (2.1) $t_d = V/Q_o$, it can be shown that, as flow decreases, and the relative change in volume is not as significant, the detention time will increase. Another characteristic of the steady-state equation is that the volume remains relatively constant, yet, the volume has significant changes as inflow and outflow rates change. These characteristics cause detention time to vary from a relatively small value at higher outflows to relatively high value at lower outflows. At the lower outflow rates, the detention time exceeds the interevent runoff period, sometimes by factors of 10 to 147. This excessive detention time is not a reasonable estimate of detention time since new stormwater will enter the pond after an interevent runoff period.

Pond Calibration

A calibration of both sites was done using field measured pond inflow and outflow rates with precipitation rates and pond volume. The residential hydrographs were calibrated using actual rainfall event and runoff data. The commercial site was calibrated using both rainfall-runoff data and an on-site fire hydrant, which allowed a very accurate estimate of inflow volume.

The calibration for the residential site was done using storm data of April 5, April 10, and May 11, 1988. The April 5, 1988 storm event distributed 1.36 inches of rainfall over the 19.2 acre watershed in 8 hours and 15 minutes. Examining the runoff volumes, the computer simulation yielded a runoff coefficient (C) of 0.40 versus the field data value of 0.44. The percent directly connected impervious area used was reduced to 55% to calibrate runoff volume. Also, examining the average detention times, the simulation calculated an average detention time of 20.45 hours compared to 18.38 hours from the field data. The ten percent difference between field and computer generated runoff coefficients and detention time are considered reasonable. However, additional calibration data were necessary.

The data from the April 10, 1988 storm event yielded a rainfall volume of 0.93 inches over 5 hours and 15 minutes. The runoff coefficient calculated by the simulation was 0.39 versus 0.38 from the field data, representing a difference of about 1%. Similarly, the average detention time calculated by the computer was 22.05 hours, whereas the field data yielded 20.73 hours, representing a difference of about 6%. The directly connected impervious area of 4.64 acres was not changed from the calibration using the April 5 data. The total watershed area remained at 19.2 acres.

Another storm event of May 11, 1988 was used to check the calibration for rainfall excess and pond outflow operation. The volume of rainfall was 0.45 inches in 2 hours and 45 minutes. The runoff coefficient calculated by the simulation was 0.31 compared to exactly the same number using field rainfall and runoff data. Also, the detention time average for both the field and simulated condition was about 50 hours.

The hydrograph shapes for the first two storms were examined next. The original estimate for time of concentration was 25 minutes and was not changed. The watershed area of 19.2 acres, the percent directly connected area of 4.64 acres, and the curve number of 65 for the pervious area also were not changed for the two storm events. The runoff calibrated hydrographs are shown in Figures 9 and 10. Field data on inflow rates were available for the April 5 and 10 storm events.

Extremely reasonable hydrograph shapes resulted using the SCS-curve number procedure to generate rainfall excess from the pervious area and direct translation of runoff from the rainfall on the directly connected impervious area. The outflow hydrographs for the pond were also examined and are shown in Figures 11 and 12. The comparison between outflow rates was not as exact as the runoff flow rates. Part of the difference can be explained by the fact that during the April 5 storm, the orifice of the outlet device and the weir were partially blocked causing the field data flow rate estimate to be lower than the simulated results.

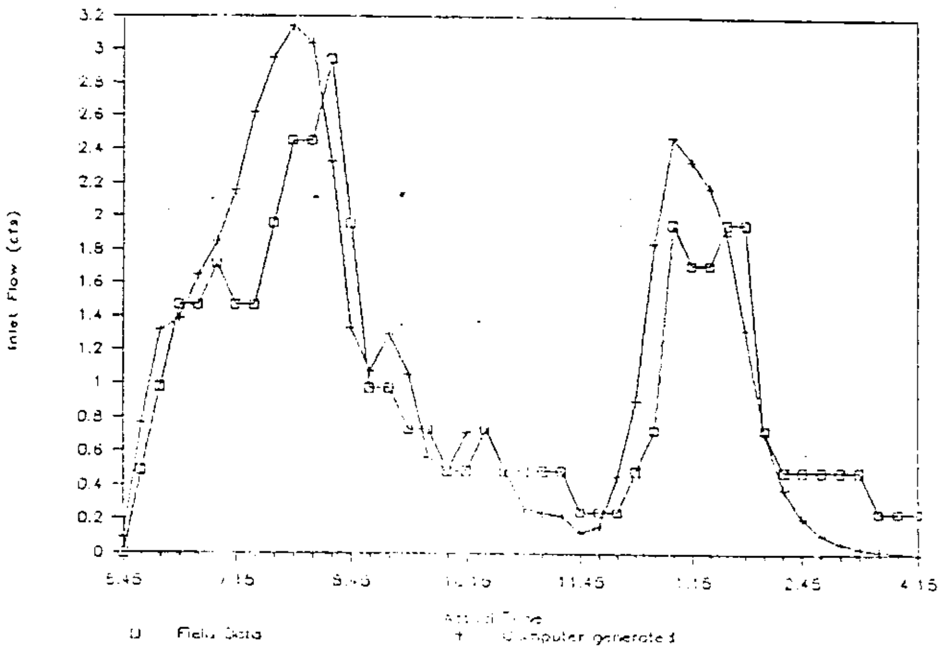

Figure 9. Runoff Calibration for the April 5, 1988 Storm Event at Residential Site.

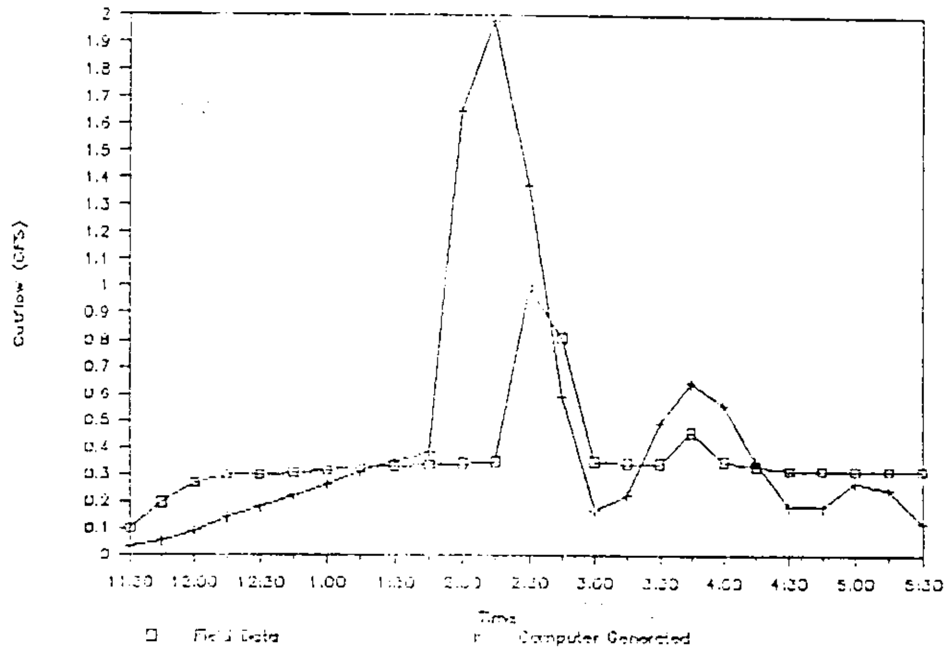

Figure 10. Runoff Calibration for the April 10, 1988 Storm Event at Residential Site.

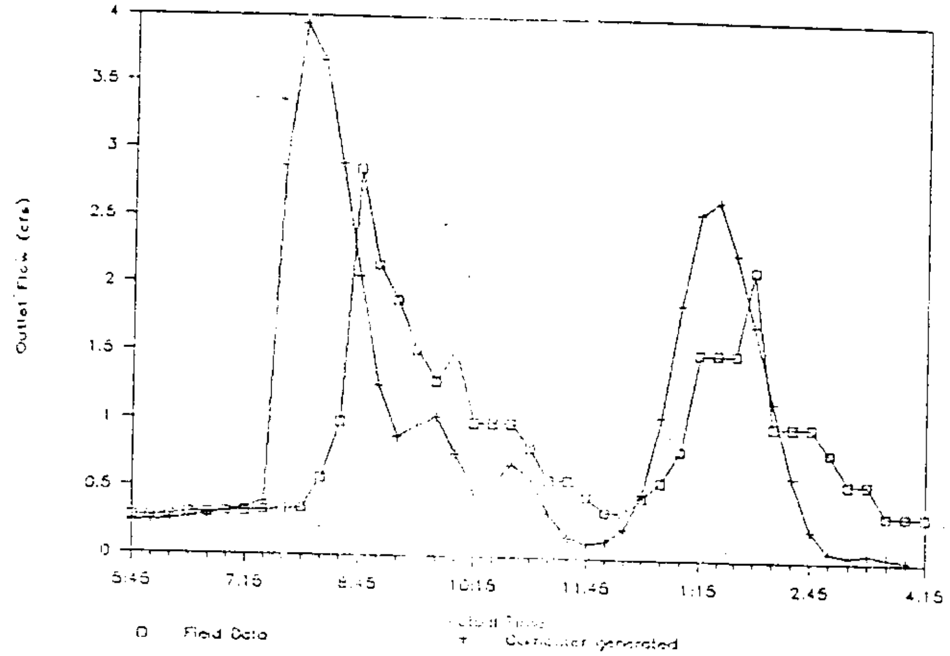

Figure 11. Pond Outflow Calibration for the April 5, 1988 Storm Event at Residential Site.

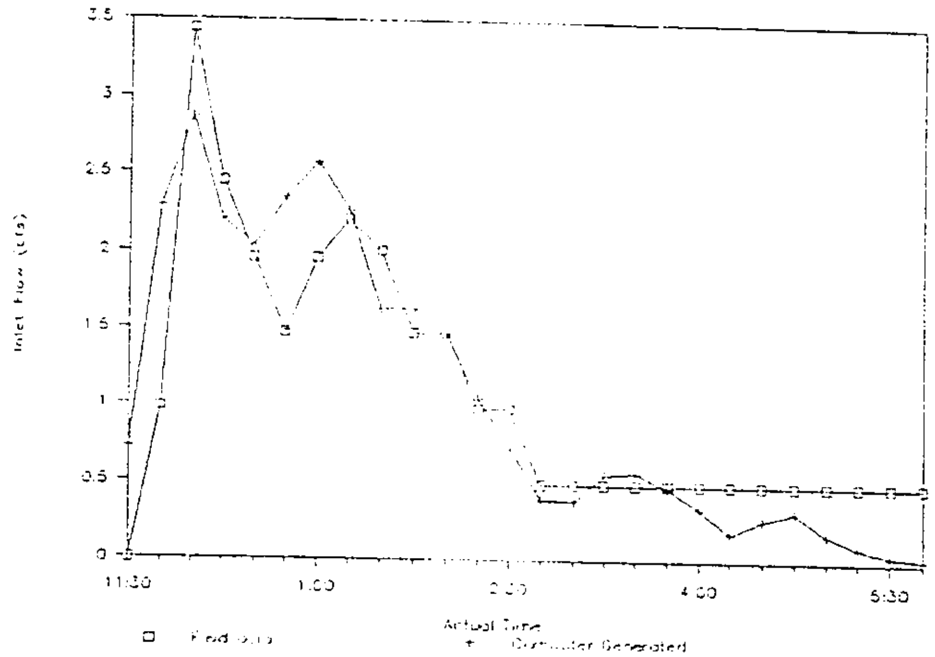

Figure 12. Pond Outflow Calibration for the April 10, 1988 Storm Event at Residential Site

Graphically, the inlet flow from the field and the inlet flow for the simulated condition show a unique compatibility, both indicate the same trends. More specifically both have significant peaks and rising and falling limbs. Similarly, the outlet flow for the simulated and the field data indicate the same uniqueness. Both plots illustrate significant peaks, the only distinguished difference is that the simulated data leads the field data by approximately 10-20 minutes. This could have been caused by a couple of factors. First, the time of concentration may have varied from the 25 minutes used. Secondly, and more likely, is in the analysis of the field data the clock timing the event may have been off by a few minutes or more due to the fact that the field data collection intervals are 15 minutes, and even though data may be taken in the interval it may have occurred at the extreme of the interval and thus it would be exaggerated in the outcome.

Using the calibrated model input parameters, a reasonable agreement between runoff coefficients, detention times, and hydrograph shapes was obtained. The mathematical model was then used to simulate a year of rainfall data to determine runoff hydrographs and detention times.

Groundwater Conditions - Residential Site

Estimates for pond outflow rate and volume using the simulation model can be significantly different if there are significant groundwater inputs or removals from the pond (Wanielista, Yousef, and Boss 1988). For a 3.2 acre detention pond with runoff from approximately a 60 acre impervious highway area, the groundwater comprised about 75% of the outflow from the pond. This caused significant reductions in the mass removal effectiveness of the pond (Wanielista, Yousef, and Boss 1988).

Groundwater was monitored at the residential site on a weekly basis. These data are shown in Figure 13. There is most likely groundwater entering the pond from the south and exiting to the north because the groundwater south of the pond is higher than the pond water and lower to the north of the pond (see Figure 13). The pond water always returned to outlet control elevation and remained at or a few inches below it before the next storm event. At the residential site, all of the simulations assumed that the pond water level was at the control elevation at the start of runoff. However, the actual field situation was that the level was a few inches less than the control. Thus, the simulated detention time values are most likely less than the actual.

Modifications to Meet FDER Standards

To meet the FDER design criteria for permanent pool, it was necessary to measure as closely as possible the acreage of impervious area. Furthermore, if only a small fraction of the impervious area were contributing runoff, the permanent pool volume based on total impervious area would be overestimated because the volume is calculated using 2.0 inches of runoff from the total impervious area and 0.5 inches from the pervious area. Other methods for calculating the permanent pool volume are to use the runoff from 3 inches of rainfall which would require an estimate of a runoff coefficient, or other infiltration calculation for the pervious area with estimates for

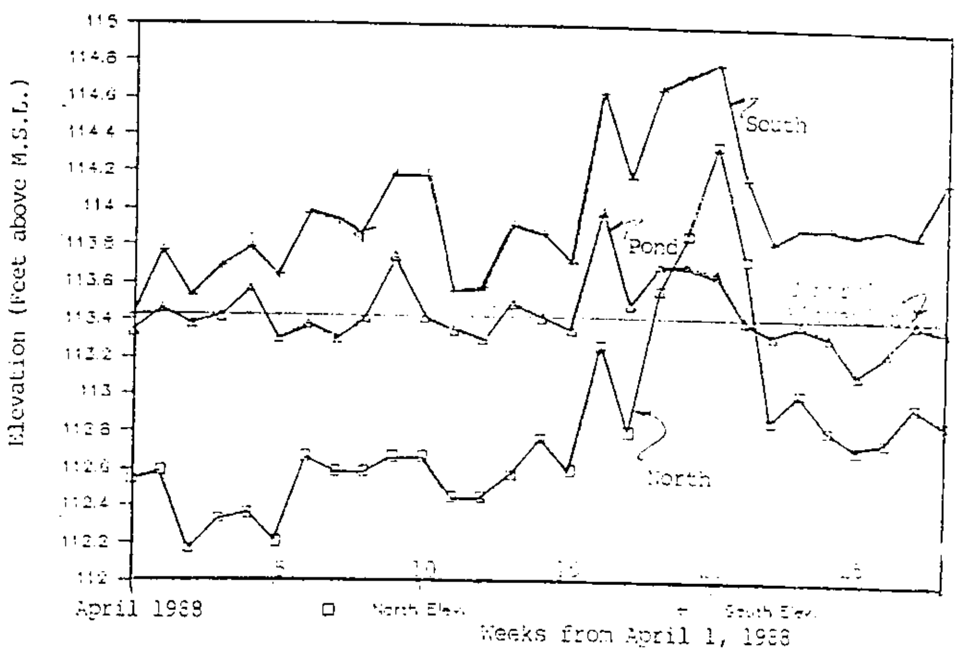

Figure 13. Groundwater and Pond Elevations at the Residential Site.

directly connected impervious areas. Thus, by field inspection of the watersheds, the directly connected impervious areas for the residential and commercial sites were estimated at 4.64 acres and 0.95 acres respectively. During two rainfall events at the residential site (0.36 and 0.45 inches) the contribution of excess from the pervious areas appeared to be minimal. Also, immediately after these storms, a double ring infiltrometer was placed in a lawn area and the limiting infiltration rates were about 10 inches/hour. This illustrates a highly permeable condition.

For the computer simulations, the modified and existing permanent volumes were used, in addition to calculated volumes using 3 formulas:

1. a. Runoff from 3 inches: Residential site with a runoff coefficient of 0.40 as measured from a rainfall of 1.36

inches at the site. For rainfall recurrence intervals of less than once in 10 years, C varies from 0.25 to 0.40 (Wanielista, 1990).

Volume = 3 inches x 19.2 acres x 0.40

x 43,560/12 (ft^2/acre)(ft/in)

= 83,635 ft^3

b. Commercial site with a runoff coefficient of 0.98.

Volume = 3 inches x 0.95 acres x 0.98 x 43,560/12

= 10,139 ft^3

2. DER Criteria: Runoff from 2 inches on impervious area plus 0.5 inches on pervious area.

a. Residential

Volume = 2 inches x 8.45 acres x 43,560/12

\+ 0.5 inches x 10.75 acres x 43,560/12

= 80,858 ft^3

b. Commercial

Volume = 2 inches x 0.95 acres x 43,560/12

= 6,897 ft^3

3. 2" DCIA criteria: Runoff from 2 inches on directly connected impervious and 0.5 inches on remaining pervious area.

a. Residential

Volume = 2 inches x 4.64 acres x 43,560/12

\+ 0.5 inches x 14.56 acres x 43.560/12

= 60,113 ft^3

b. Commercial

Volume = 2 inches x 0.95 acres x 43,560/12

= 6,897 ft^3

For the commercial site, comparing the three methods for calculating permanent pool, the runoff from 3 inches of rainfall produced the greatest volume primarily because there was an insignificant amount of pervious area. This compares to the present design of 0.50 inches over the total area. This increase in volume would not cause flooding of the watershed, but additional area would be needed.

For the residential site, comparing the three methods for calculating permanent pool, the runoff from 3 inches over the total watershed and the DER criteria produced similar results, while the 2" over the directly connected impervious area produced a lower volume (approximately 60,000 cubic feet compared to 80,000 cubic feet). The increase in pond size and depth of permanent pool to 5 feet with an invert of the discharge at elevation 113.40 would not cause flooding in the watersheds. This was verified by field survey with a hydraulic profile shown in Figure 14. Note that the roadway gutter has a minimum elevation of 115.90. The runoff from the 25 year storm event (6 inches) did not force the hydraulic profile into the street.

The volume of stormwater discharged over time must meet the DER criteria, which states that no more than 50% of the volume of live storage for the runoff from 1" of rainfall can be released in less than 60 hours. To meet this criteria, a 3 inch orifice was used. The runoff volume was estimated at 28,000 cubic feet with a maximum storage depth of 1.45 feet. The orifice equation used is:

$$Q = 0.6(\ D^2/4)(2gH)^{1/2} \qquad 0\leq H\leq 1.45' \qquad (12)$$

where: Q = Discharge rate, cfs

D = Orifice diameter of 3 inches, or 0.25 feet

H = Head on the center line of the orifice, feet

when the orifice was not submerged, a weir equation was used. For conditions above 1.45 feet, a 90° V-notch weir was used.

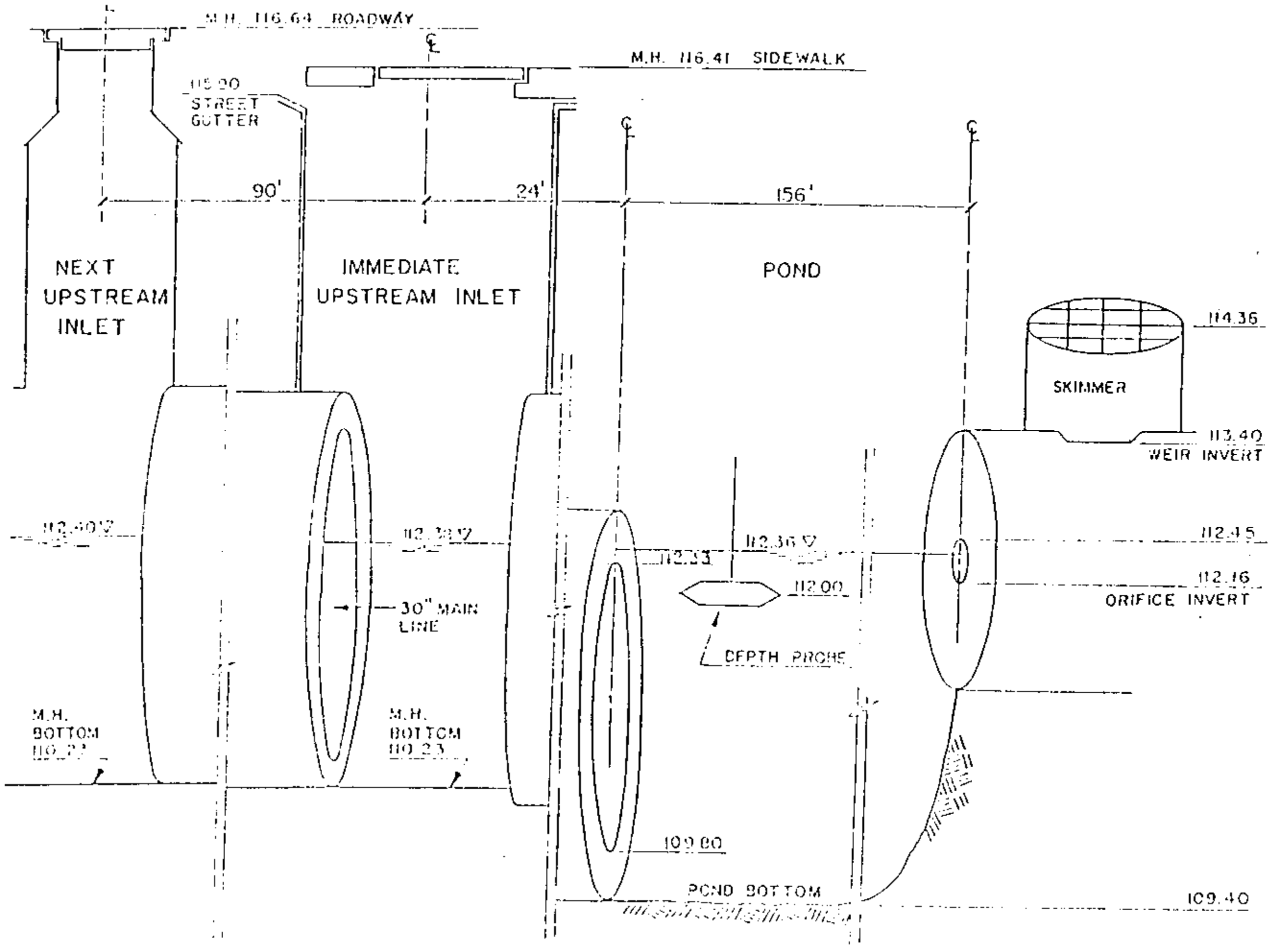

Figure 14. Pond and Sewer System Hydraulic Profile for the Residential Site.

CHAPTER 5

RESULTS

The presentation of the results is to primarily compare frequency distributions for detention times using six different design criteria. Four design criteria for the permanent pool of a detention pond at the residential site and two design criteria for permanent pool of a detention pond at the commercial site are compared. These frequency distributions are compared to the frequency distribution for time between runoff events (inter-event times). The frequency distributions were developed using one year of rainfall data to produce pond inflow rates, and a pond inventory equation with a discharge relationship to estimate pond outflow and volumes. Frequency distribution for detention time data include less than and exceedence types.

The 1985 calendar year rainfall data was used for the simulation. This year was choosen because from 9 previous years (1979-1987) it represents most closely the average conditions as measured by rainfall statistics and shown in Table 4. For the year 1985, there were 62 rainfall events with volume greater than or equal to 0.10 inches and 88 rainfall events with volume greater than or equal to 0.04 inches. There were 12 events with rainfall volume greater than one inch. The maximum rainfall volume for a storm event was 4.12 inches. The duration of a rainfall events ranged from 15 minutes to 20 hours with an average of 3.95 hours.

TABLE 4

COMPARISON OF RAINFALL STATISTICS
SIMULATION YEAR VERSUS LONG-TERM

	SIMULATION YEAR	9 YEARS (1979-1984)
Number of Storms > Trace	118	124
Number > 0.04 inch	88	91
Number > 0.10 inch	62	61
Number > 1.00 inch	12	12
Maximum Rainfall Volume (in)	4.12	5.04
Average Rainfall Volume (in)	0.52	0.51
Average Duration (hrs) for rainfall > 0.04 inches	3.95	4.28
Average Inter-Event Dry Period (hrs) for rainfall > 0.04 inches	95.10	90.60
Volume per year (in) for rainfall > 0.04 inches	45.76	46.41

Commercial Site

For the existing design at the commercial site, the detention time for surface water discharges never exceeded 18 hours with an average of 3.5 hours. The temporary storage was discharged to surface water over the weir and by bank infiltration. The bank infiltration averaged 1/4 inch per hour. A typical water table fluctuation curve is shown in Figure 15. The pond elevation was usually greater than the groundwater

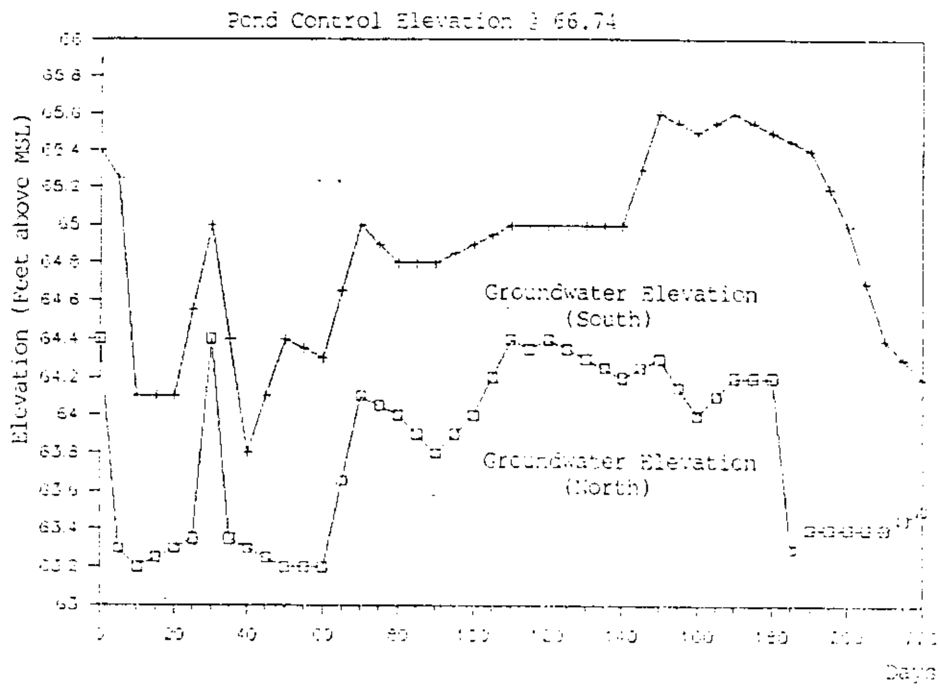

Figure 15. Water Table Fluctuation at the Commercial Site.

elevation. The commercial site detention time is a relatively low value, however, the primary discharge (about 85% of the outflow volume) is by bank infiltration. This may provide an effective means for water quality improvement.

When the pond volume was increased to store the runoff from 3 inches over the watershed, the pond depth increased to 8 feet to fit into the available area. However, the pond volume decreased by bank infiltration to about 3 feet below the weir between runoff events making the long term depth of pond about 5 feet.

With an average bank infiltration rate of 1/4 inch per hour, the live volume above the permanent pool would take a maximum of 96 hours to drain. Frequency distributions for detention time (for surface water discharge) had an average of about 3.8 hours with the maximum never exceeding 14 hours. The additional permanent pool storage did not significantly change the frequency distribution (see Table 5). The probable reason is that about 85% of the runoff water is treated by infiltration. Thus, detention time for surface outflow is not greatly changed.

Residential Site

The evaluations and results for the residential site also were based on detention time using surface water discharges. First, 88 rainfall events with rainfall greater than 0.04 inches were simulated. For rainfall volumes greater than 0.04 inches, it was assumed that runoff would result.

TABLE 5

COMPARISON OF DETENTION TIME FREQUENCY DISTRIBUTIONS FOR 1/2" AND 3" RUNOFF DESIGNS AT THE COMMERCIAL SITE BASED ON SURFACE WATER DISCHARGE

DESIGN PERMANENT POOL RUNOFF VOLUME (inches)	DETENTION TIME AVERAGE (hours)	MEDIAN (hours)	MAXIMUM (hours)
1/2"	3.5	2.1	18
3"	3.8	2.0	18.4

Depression storage within the impervious area may be significant enough to limit the runoff to rainfall conditions of greater than 0.10 inches. From on-site observation of an 0.03 inch and an 0.11 inch storm, runoff did occur for the 0.11 inch rainfall, but no flow in the sewer was noted for the 0.03 inch rainfall.

The "as-built" permanent pool and discharge structure were used to estimate a frequency distribution for detention time. This first comparison was done using outflow rates and volume to calculate detention time. Once detention time was calculated at 15 minute intervals, the 2141 data points were summarized by statistical measures. The simulation model was executed a second time using criteria of rainfall events greater than 0.10 inches which lowered the number of events producing runoff to 62. This yielded 1616 fifteen minute estimates for detention times. In Table 6 is illustrates a comparison of the findings with inter-event runoff time. The inter-event runoff average time is about 50% greater than the detention time calculated.

TABLE 6

COMPARISON OF DETENTION TIMES (HOURS) USING "AS-BUILT" CONDITIONS AND RAINFALL DATA WITH MINIMUM RAINFALL CRITERIA OF 0.04 AND 0.10 INCHES

	$\geq$ 0.04 inches	$\geq$ 0.10 inches	INTER-EVENT RUNOFF* "t_o"
NUMBER OF STORMS	88	62	88
MEAN	65.61	66.46	99.05
MEDIAN	55.00	52.50	75.10
STD DEVIATION	67.37	66.98	120.31

* based on rainfall $\geq$ 0.04 inches

Since the runoff from the rainfall of all events greater than or equal to 0.10 inches produced the lowest detention values, it was decided to limit the simulations to rainfalls of 0.10 inches or greater. The data illustrates the type of distribution for detention time. Since the mean value is greater than the median value, a right-skewed distribution would most likely fit the empirical data.

Detention time distributions were compared using four design criteria for the permanent pool. The program limited the maximum detention time to be equal to the time between outflow events. The criteria was developed using rainfall volumes $\geq$ 0.10 inches. Both outflow and average of inflow and outflow were used to estimate the frequency distribution for detention times. The computer program, using specified time period of analysis, calculated average detention times for each storm. The Weibull plot position was used to develop an empirical distribution for detention times using both detention time formulas (outflow and average of inflow and outflow). Both results were almost identical. The same general trends were observed for all three designs. Figures 16, 17, and 18 illustrate the frequency distribution for three design conditions and as calculated using both detention time formulas. The form of the frequency distributions is similar to that for an intermediately mixed pond (Levenspiel 1962).

An average detention time for each rainfall event for 62 events per year was calculated from the 15 minute estimates of detention time using both detention time formulas. The results are shown in Table 7. From the comparison, the minimum average event detention time was estimated to be 21 hours, while the maximum was 700 hours. Comparing the estimates using both formulas, for detention time, the detention calculated using outflow ratesproduced lower estimates which were considered to be a more conservative estimation and would be used for comparing frequency distributions. The resulting frequency distributions should be right skewed (positive skewness) because the average value always exceeded the median value.

Comparison of the averages and median values for each design criteria indicates that detention time increased with increasing permanent pool. Using DER criteria (or the runoff from 3 inches of rainfall) the highest average (88 hours), and highest median (74 hours) values resulted.

TABLE 7

COMPARISON OF DETENTION TIME STATISTICAL PARAMETERS
USING AVERAGE DETENTION TIMES PER STORM EVENT
(all values in hours)

					AVERAGE OF OUTFLOW AND INFLOW			
	AS-BUILT	OUTFLOW MODIFIED	2" DCIA*	DER	AS-BUILT	MODIFIED	2" DCIA	DER
MINIMUM	21	22	24	24	21	22	24	24
MAXIMUM	230	359	542	671	266	390	683	700
AVERAGE	66	75	84	88	74	80	85	92
MEDIAN	52	56	73	74	56	65	69	78

* DCIA = Directly Connected Impervious Area

A comparison of the empirical cumulative frequency distributions on detention time are made using detention time based on outflow rates for specified detention times. The comparison is shown in Table 8.

These frequency distribution values were obtained from the empirical data and can also be estimated from Figures 16, 17, and 18. For the "as-built" situation, 68% of the time, a detention time of 72 hours or less in obtained. However, only 46% of the time, the time between outflow events is 72 hours. If the permanent pool is increased to the runoff from 2 inches of rainfall over the directly connected impervious area (2" DCIA), then only 48% of the time, detention time is less than or equal to 72 hours. For the DER criteria (or runoff from 3 inches of rainfall), detention time less than or equal to 72 hours is further reduced to 47% of the time, and closely approximates the time

TABLE 8

COMPARISON OF THE EMPIRICAL CUMULATIVE DISTRIBUTION DATA FOR DETENTION TIMES USING FOUR DESIGN CRITERIA WITH DETENTION TIME BASED ON TIME BETWEEN OUTFLOW EVENTS

DETENTION TIME (hrs)	LESS THAN AS-BUILT	MODIFIED	2"DCIA	DER	t_o
48	0.44	0.43	0.35	0.35	0.35
72	0.68	0.60	0.48	0.47	0.46
96	0.80	0.76	0.67	0.63	0.61
168	0.91	0.89	0.89	0.84	0.81
336	0.99	0.99	0.95	0.92	0.89

DETENTION TIME (hrs)	EXCEEDENCE AS-BUILT	MODIFIED	2"DCIA	DER	t_o
48	0.56	0.57	0.65	0.65	0.65
72	0.32	0.40	0.52	0.53	0.54
96	0.20	0.24	0.33	0.37	0.39
168	0.09	0.11	0.11	0.16	0.19
336	0.01	0.01	0.05	0.08	0.11

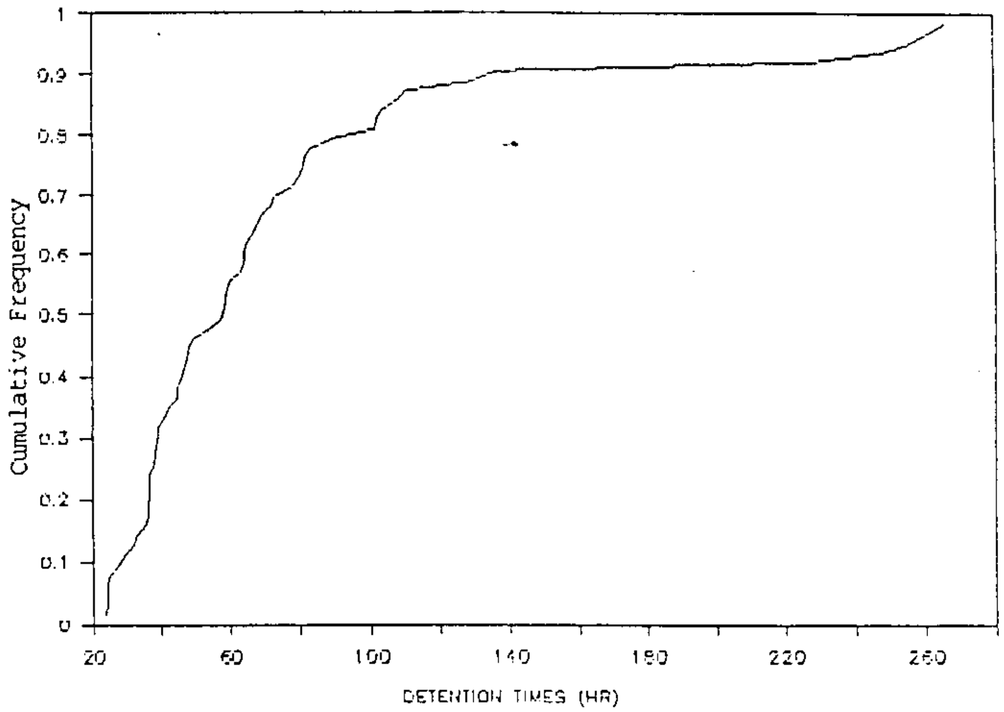

Figure 16. Empirical Distribution - Residential "As-Built" Outflow Structure - Using Outflow Rate.

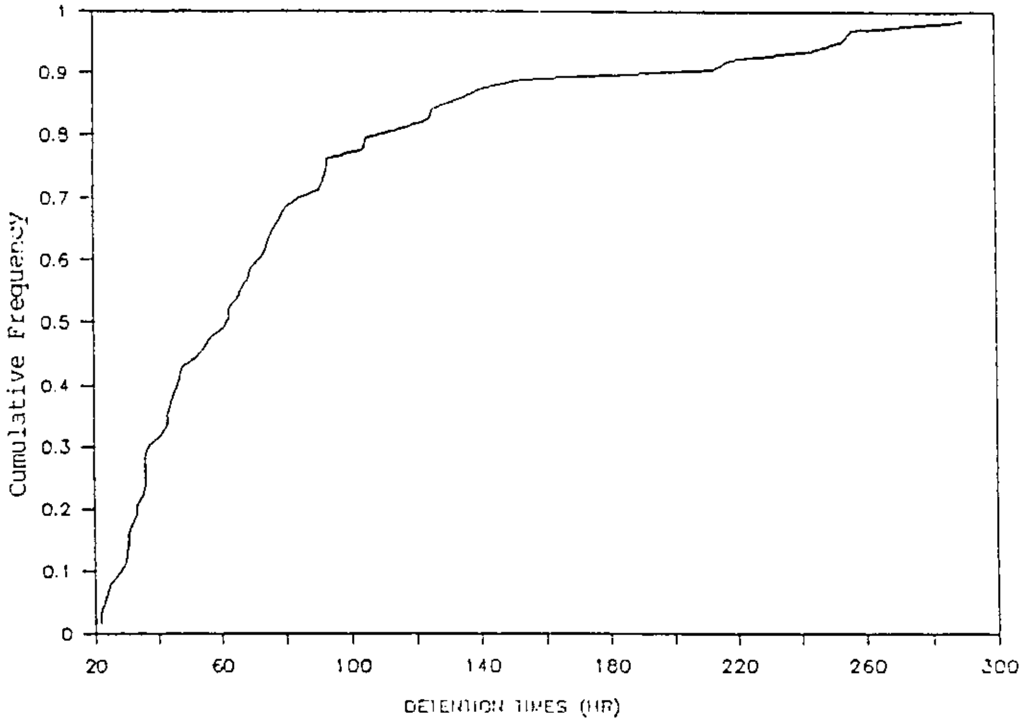

Figure 17. Empirical Frequency Distribution - Residential Modified Outflow Structure - Using Outflow Rate.

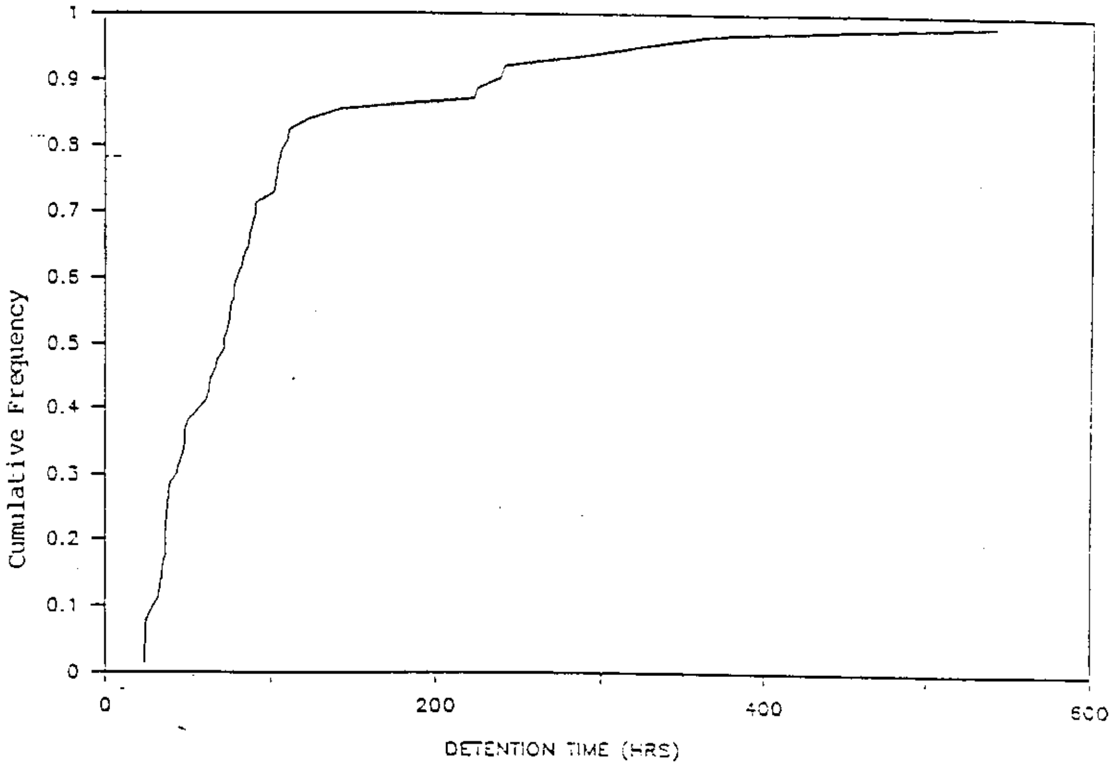

Figure 18. Empirical Frequency Distribution - Residential DER Criteria Outflow Structure - Using Outflow Rate.

between outflow events. Other detention times are compared with similar results. The DER criteria produces higher detention times at a greater frequency than the "as-built," modified, or 2"DCIA criteria.

The exceedence probability is defined as one minus the less than probability. It illustrates the percentage of times the specified detention time is exceeded. The greater the exceedence, the better the design. Again, the DER criteria has exceedence values closer to those for the time between outflow events.

Observations on estimated times to drain the pond of live storage can be made for the 62 rainfall events used for the simulation. DER (Livingston 1988) requires that no more than 1/2 of the live storage from 1 inch of runoff be drained in 60 hours or more. There were only two storms that produced about 1 inch of rainfall and in one case, the time to drain half the live storage volume was about 55 hours. The other storm drained half the storage in 68 hours.

The comparisons of detention times for all frequencies is accomplished by developing a theoretical frequency distribution which best fits the empirical one. The data for three permanent pool designs (as-built, 2"DCIA, and DER) are used.

The method used to evaluate the detention times is one written by Wanielista (in press) called LEASTS which evaluates six different distributions, namely, (1) truncated normal, (2) two parameter log-normal, (3) three parameter log-normal, (4) Gumbel, (5) Pearson, and (6) Log-Pearson. The analysis of each distribution provides the following information: parameter estimation, random test statistic, standard error, equation values, residual values, and graph of the probability vs. actual and equation values. In addition, sorted recorded events are developed. The method of Maximum Likelihood is used to estimate parameters. If the method does not converge, then the Method of Moments is used.

It was found using graphical comparisons, and the Kolmogov-Smirnoff statistic that the Log Pearson Type III distribution yielded the best fit. The theoretical fit with comparison to the empirical are shown in Appendix B. The results are shown in Figure 19 and indicate that for all ranges of frequencies the DER criteria produces higher detention times and approaches the interevent outflow times.

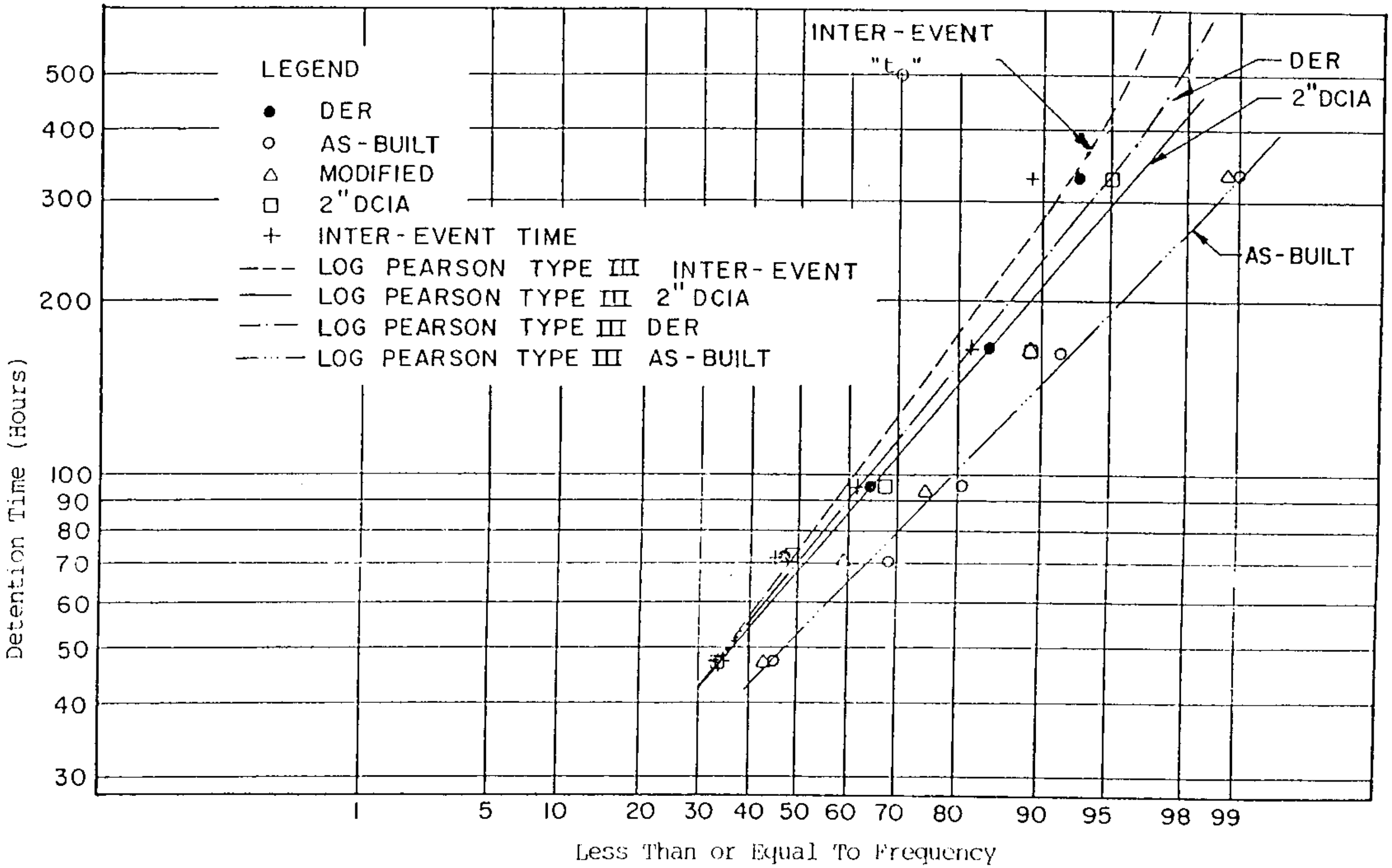

Figure 19. Comparison of Detention Time Frequency Distributions Using a Log-Pearson Type III Theoretical Distribution.

CHAPTER 6

SUMMARY AND RECOMMENDATIONS

The purposes of this research were to develop frequency distributions for detention time using different permanent pool design volumes, then to compare these distributions to distribution on time between outflow events, which is a measure of maximum detention time. The result is a specification on stormwater detention pond designs for which a detention time frequency distribution is approximately equal to that of a distribution for time between pond outflow events.

Detention ponds that were operational for two watersheds were used to collect watershed and pond inflow, outflow, and volume data. The results are limited to four permanent pool design volumes with outflow controls for the residential site and two permanent pool design volumes for the commercial site. Total pond volume was assumed to be reactive or there was no short-circuiting in each pond. By using total pond volume, the estimated frequency distribution for detention time is actually greater than that expected with a smaller reactive volume. However, near total pond volume should be used and can be achieved by proper geometric design and properly routing flow through the pond.

Field collected data were obtained for inflow rates and outflow rates with changes in pond volume for the two stormwater detention facilities. The commercial site inflow rates and bank infiltration rates were calibrated using flows from an on-site fire hydrant and pond depth changes with time, whereas the residential site inflow rates were calibrated using pond depth changes, rainfall volumes and inflow velocity measures since the watershed was not all impervious and relatively large. The discharge structures were calibrated using direct volume discharge measurements. Equations were then fit to the discharge data using the method of least squares. For the commercial site, bank infiltration was estimated using pond volume changes over time.

A simulation model was calibrated for inflow rates, outflow rates, pond volumes, and detention time. One year of rainfall data were used to simulate runoff from the watersheds as input to the detention ponds. The SBUH hydrograph generation method was used. The pond volume and outflow rates were simulated using inventory equations and pond volume discharge relationships. These data on inflow rates, outflow rates, and pond volume are used to calculate detention time.

The detention times were calculated on a 15 minute time interval. Frequency distributions were developed for the 6 designs. A statistical analysis was used to determine the theoretical frequency distribution for detention time. The distributions for various permanent pool volumes were compared to a distribution for inter-event runoff. The inter-event time period represented the standard. The frequency distributions followed those of an intermediately mixed pond.

Recommendations

Higher detention times for stormwater detention ponds can be achieved using larger permanent pools and longer pond drainage times.

This will result in frequency distributions for detention times that approximate the frequency distributions for time between pond outflow events. The largest permanent pool volume results when calculated using the specification of the runoff volume from 3 inches of rainfall or other suitable calculations that produce similar runoff volumes. This specification produces a frequency distribution that approximates the frequency distribution for time between outflow events. Using the DER specifications of 2 inches times the total impervious area plus 0.5 inches times the pervious area approximates the criteria of runoff from 3 inches of rainfall for the residential site but when compared to the commercial site is less by 1" over the impervious watershed. It is recommended that the runoff from 3 inches of rainfall be used to size the permanent pool of a detention pond. Additional permanent pool volume will not increase detention time significantly because the maximum detention time has an upper limit set by the time between outflow events.

To achieve relatively higher detention times, maximum use of pond volume should be achieved. This is achieved by reducing short-circuiting in the pond. Baffles can be used to increase the path of flow and hence the detention times.

Future Work

This work introduces detention time as a method for evaluating effectiveness of stormwater detention facilities. Future work should correlate water quality data with detention times and the interevent outflow time period. The analysis should include ponds with bank infiltration systems as well as the standard detention pond design with pond control elevation approximately equal to the groundwater elevation.

For detention ponds with significant bank infiltration, detention time based on surface outflow does not provide adequate representation of effectiveness. Therefore, water quality studies should include data on the percentage of pond inflow waters that discharge by infiltration. For the commercial site, about 85% of the runoff water was estimated to discharge by means of infiltration.

When comparing detention time frequency distributions for all permanent pool design criteria, it should be noted that calculated detention time was less than 72 hours over 50% of the time. Since higher efficiencies depend on longer detention times a greater percentage of the time, there will be failures to meet removal efficiencies associated with 72 hour detention time, a majority of operating time. A minimum detention time should be specified to achieved a desired removal efficiency. It is recommended that this minimum time be equated to an inter-event dry period. Once the minimum time is established, a conditional probability distribution for rainfall volume given a minimum inter-event dry period (no rainfall) can be developed. These conditional probabilities can be expressed as precipitation volume, inter-event dry period, frequency (PIF) curves. The detained runoff waters can be held for the specified minimum detention time to achieve a desired level of treatment.

APPENDIX A

GRAIN SIZE DISTRIBUTION CURVES

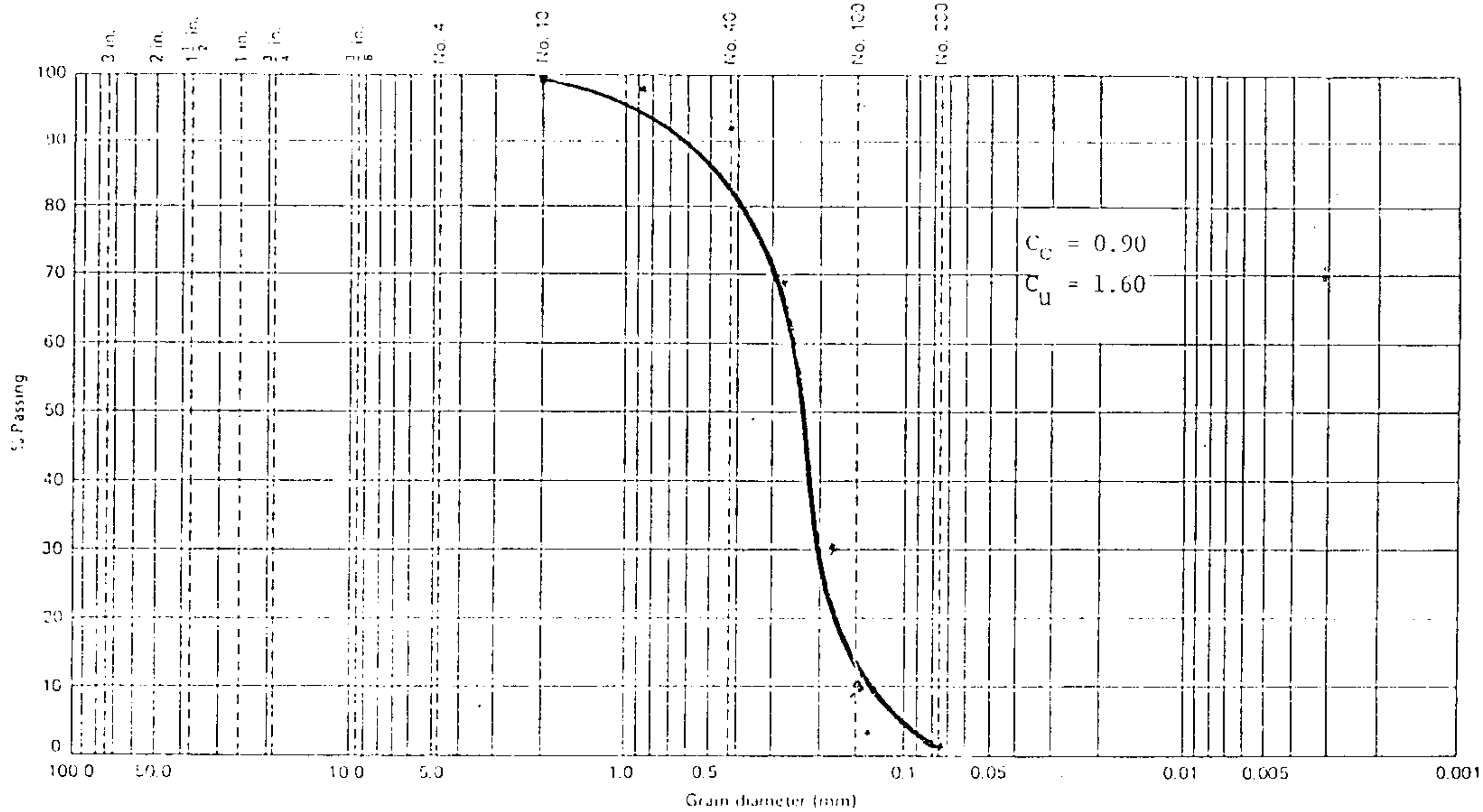

Figure 20. Commercial Site - Southwest - 3 foot depth - Pond Bank

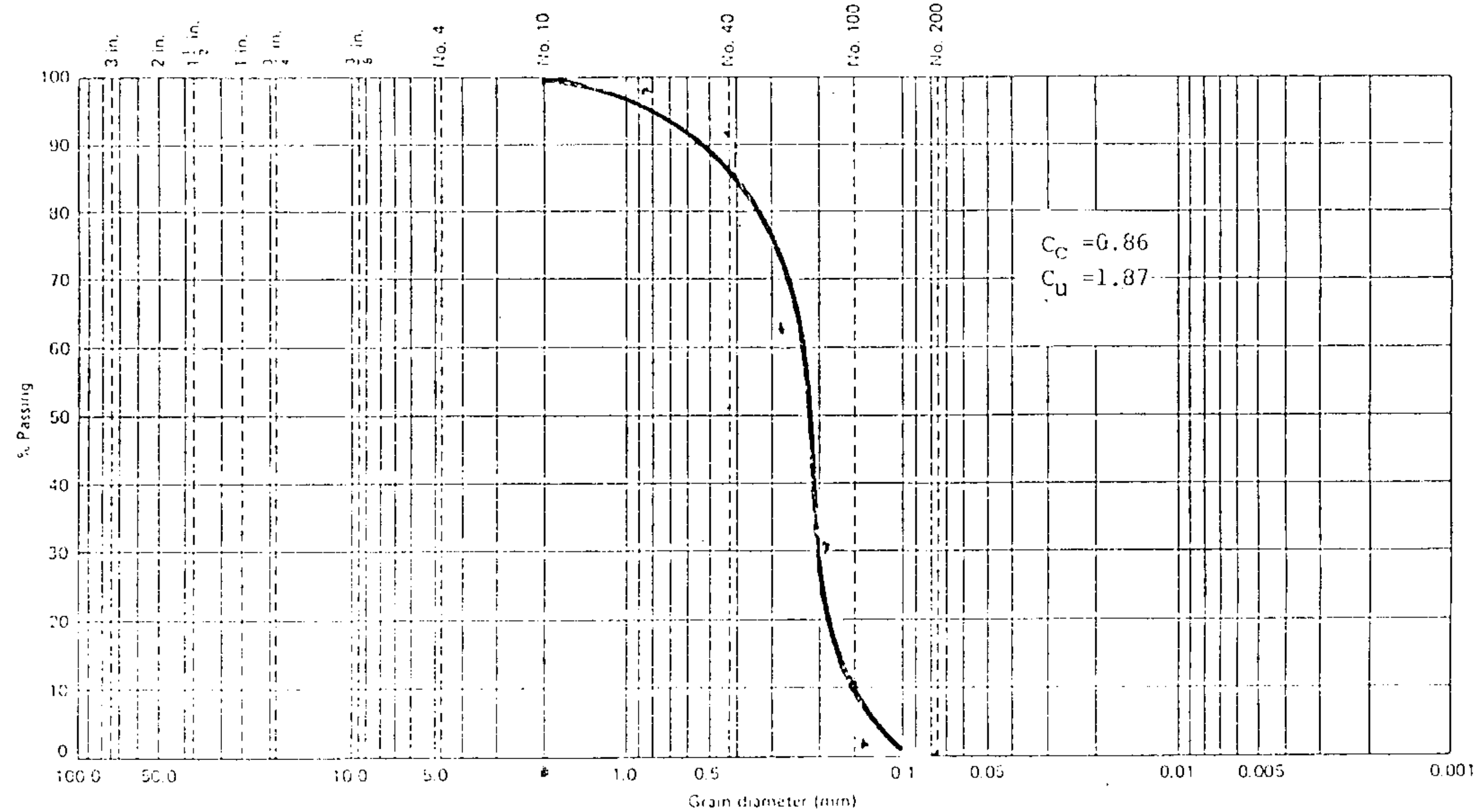

Figure 21. Commercial Site - Northeast - 3 foot depth - Pond Bank

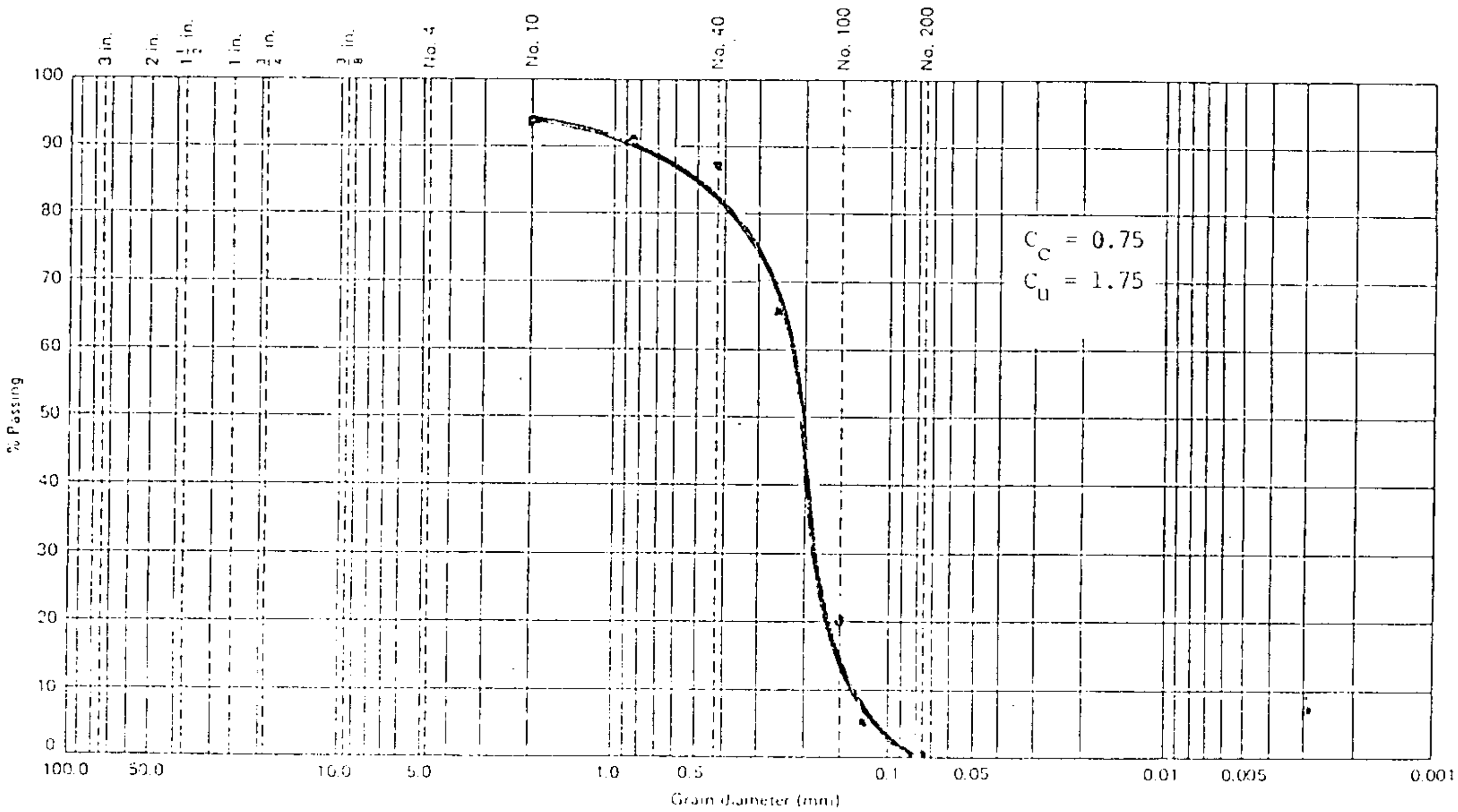

Figure 22. Commercial Site - Pond Bottom - 2 foot depth

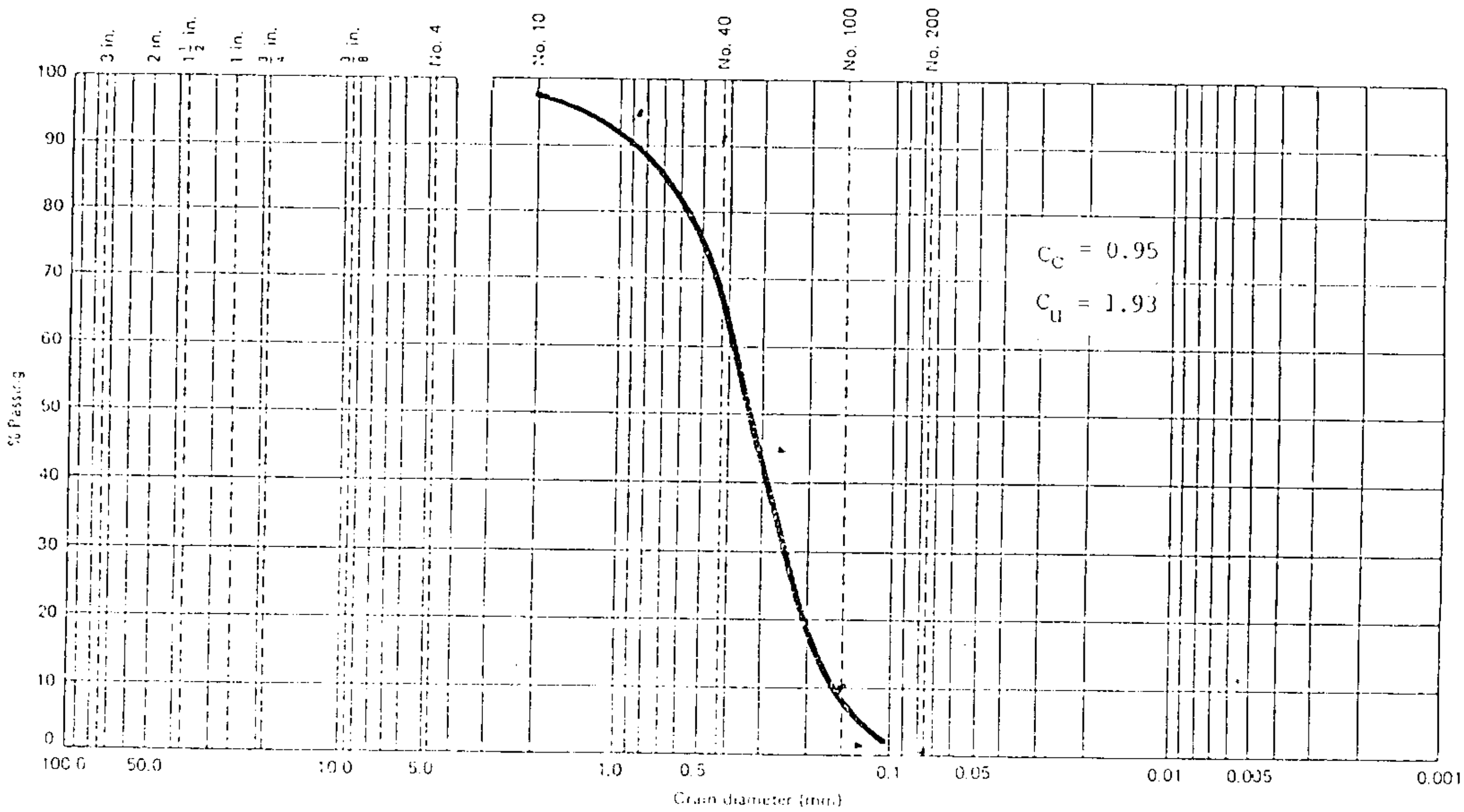

Figure 23. Residential Site - Southwest - 1 foot depth - Pond Bank.

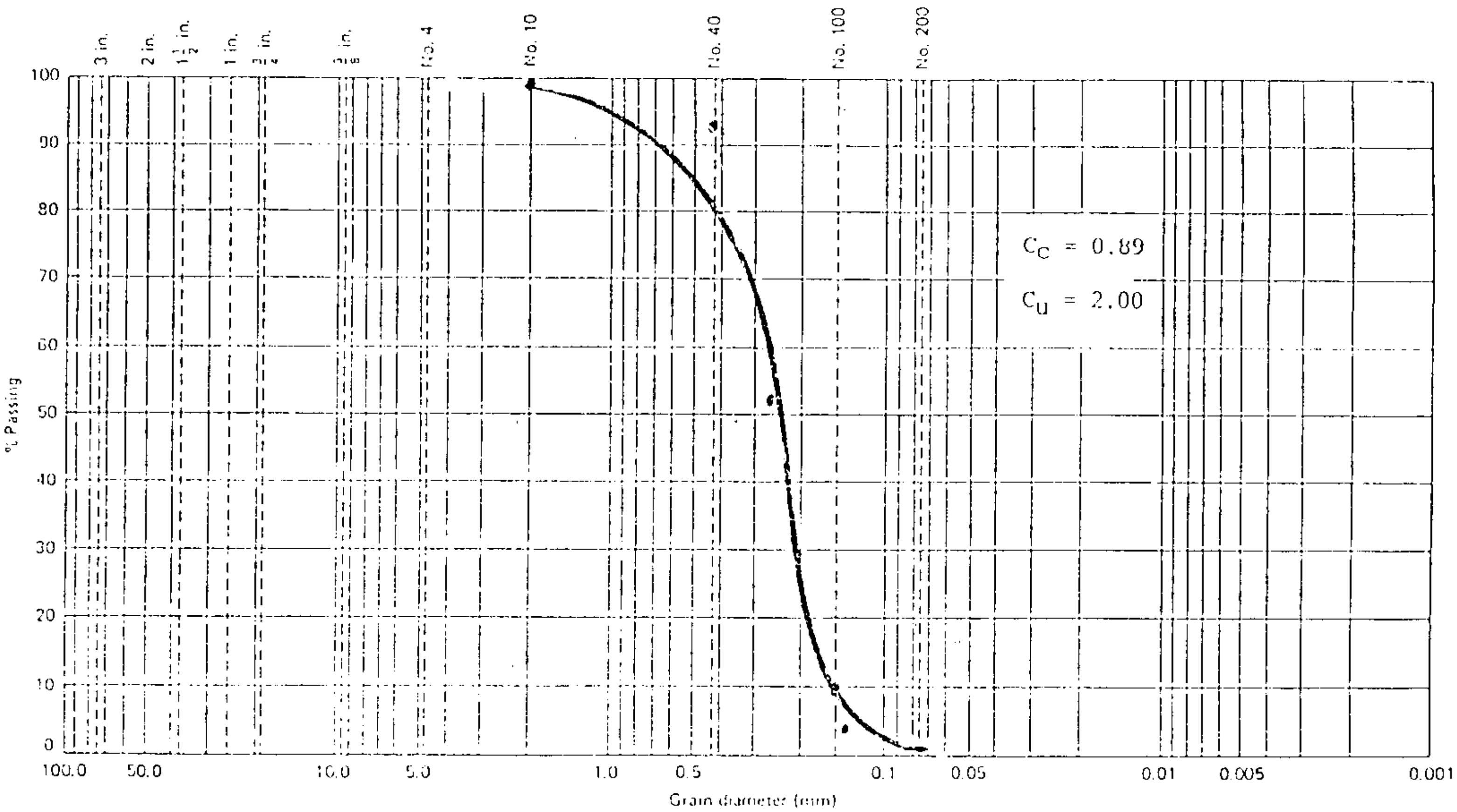

Figure 24. Residential Site - Southwest - 3 foot depth - Pond Bank.

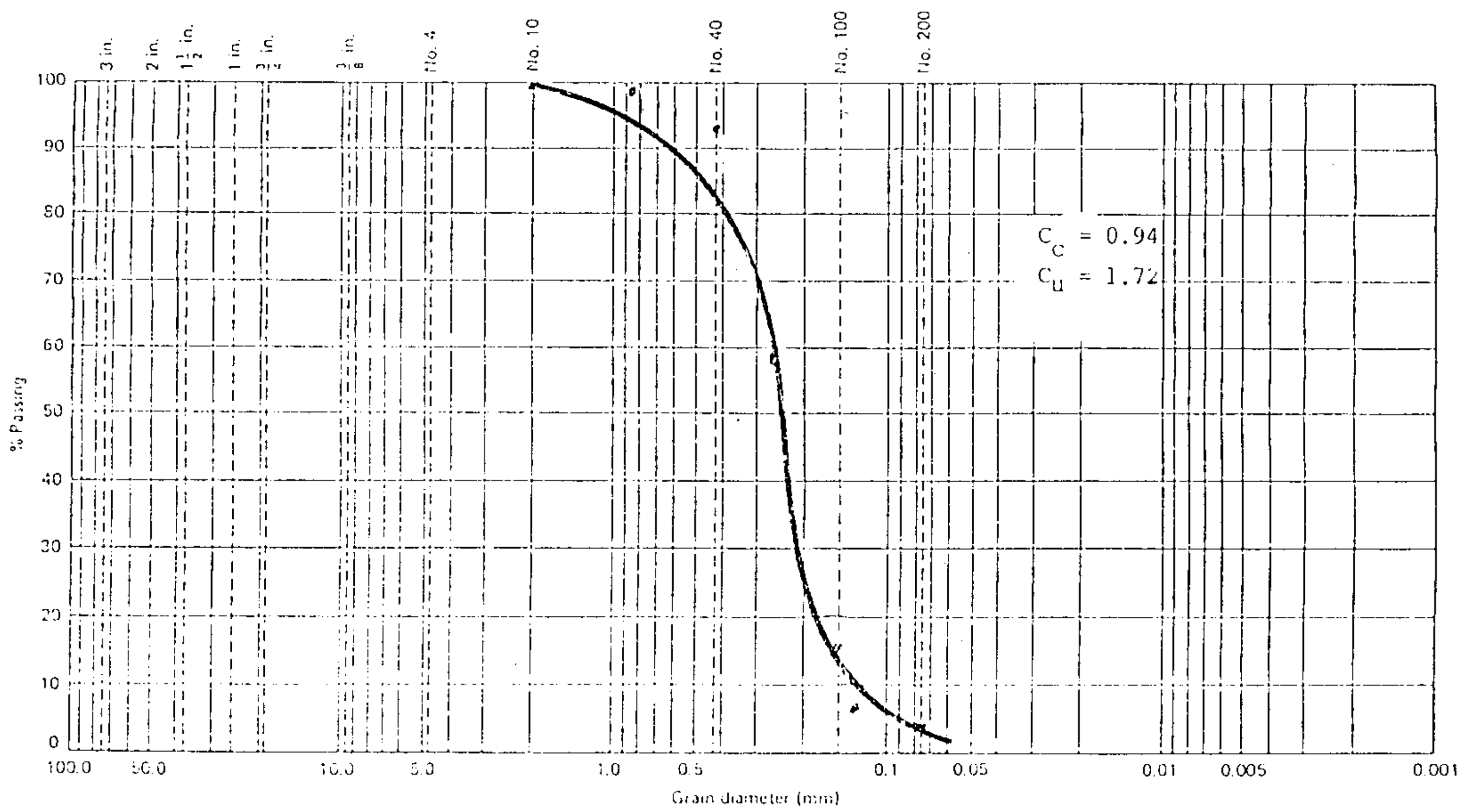

Figure 25. Residential Site - Northeast - 1 foot depth - Pond Bank.

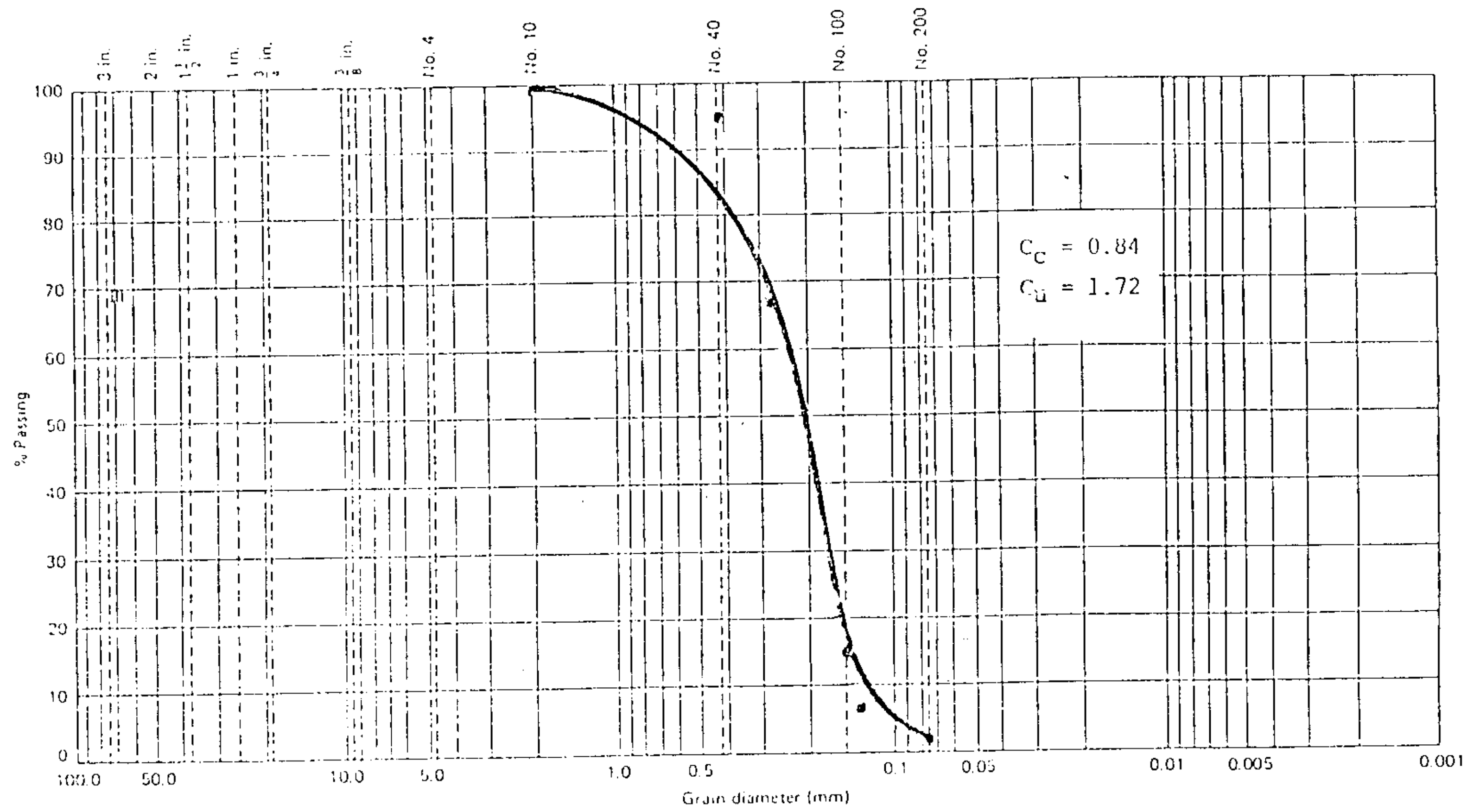

Figure 26. Residential Site - Northeast - 3 foot depth - Pond Bank.

APPENDIX B

THEORETICAL DISTRIBUTIONS vs. EMPIRICAL DISTRIBUTIONS

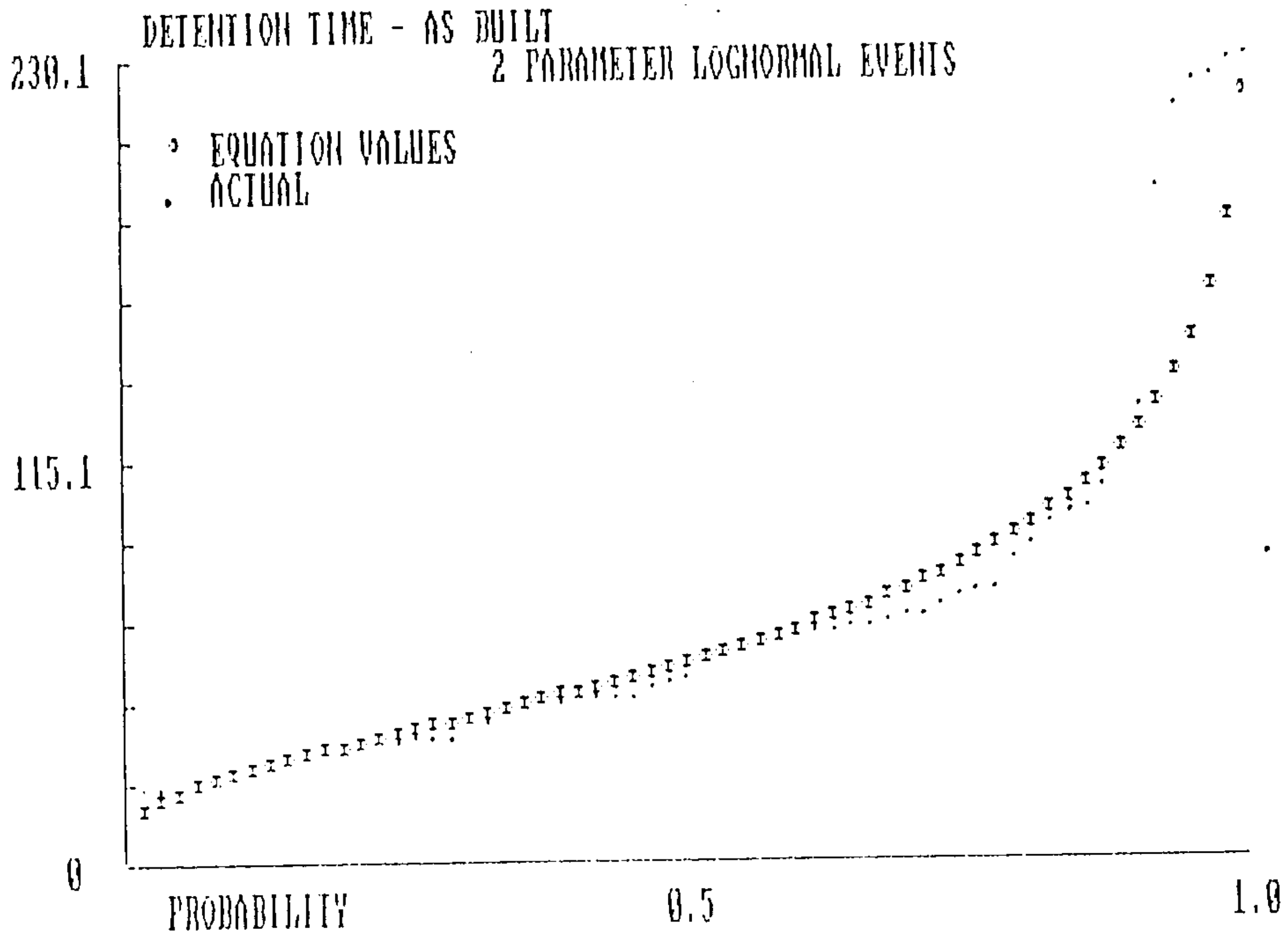

Figure 27. Log-Normal - As-Built - Detention Time Distribution.

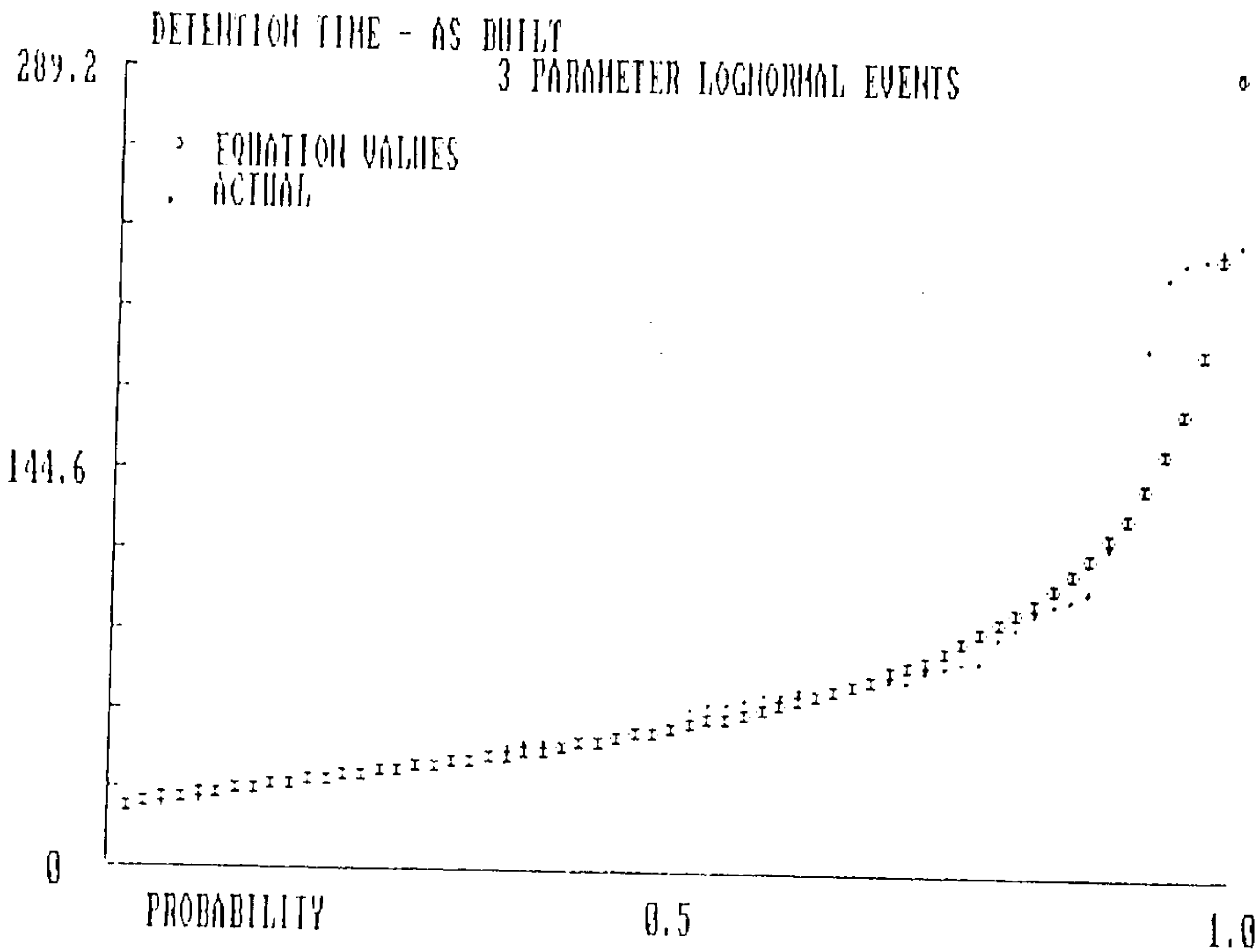

Figure 28. 3-Parameter Log-Normal - As-Built - Detention Time Distribution.

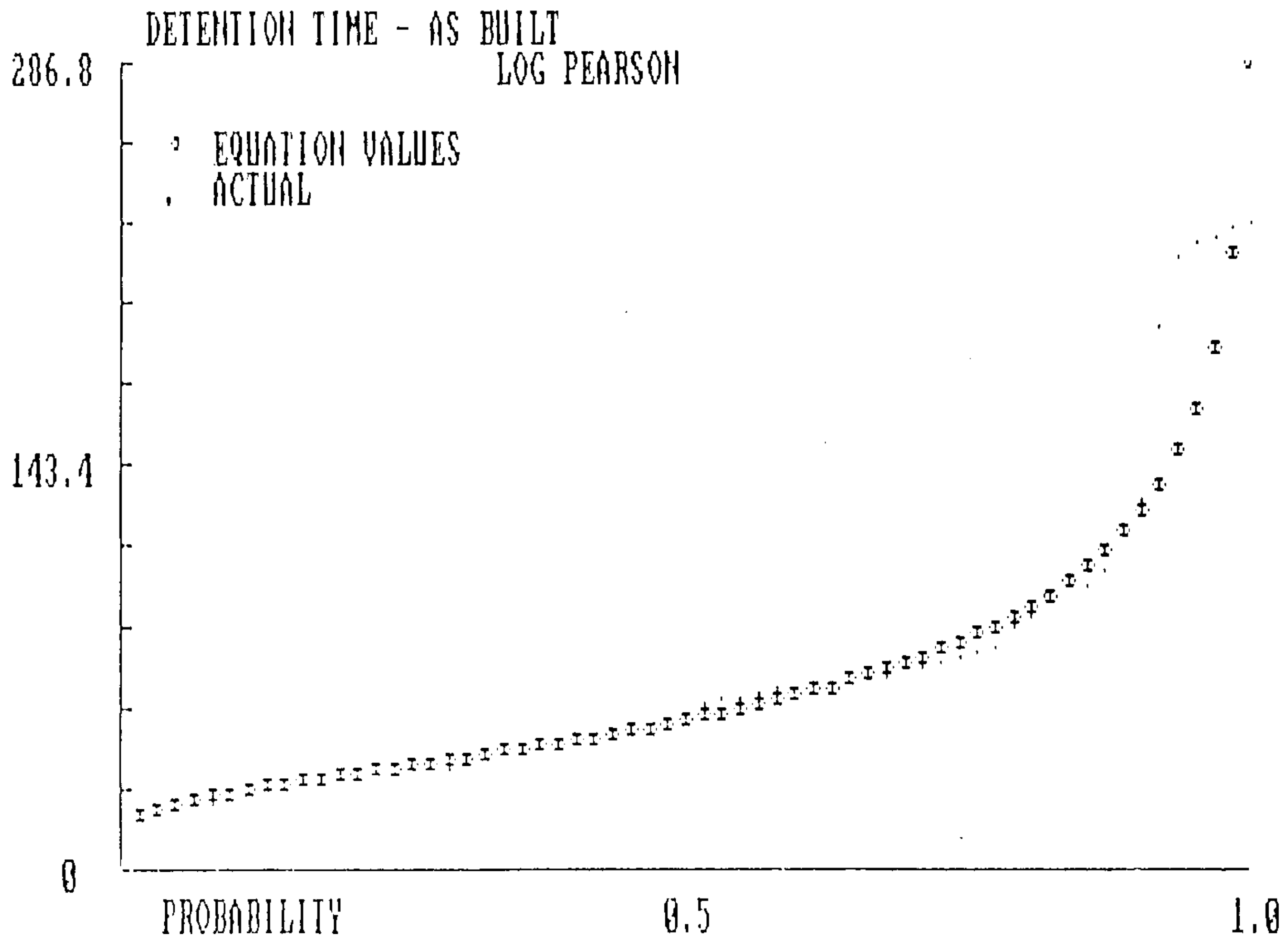

Figure 29. Log-Pearson Type III - As-Built - Detention Time Distribution.

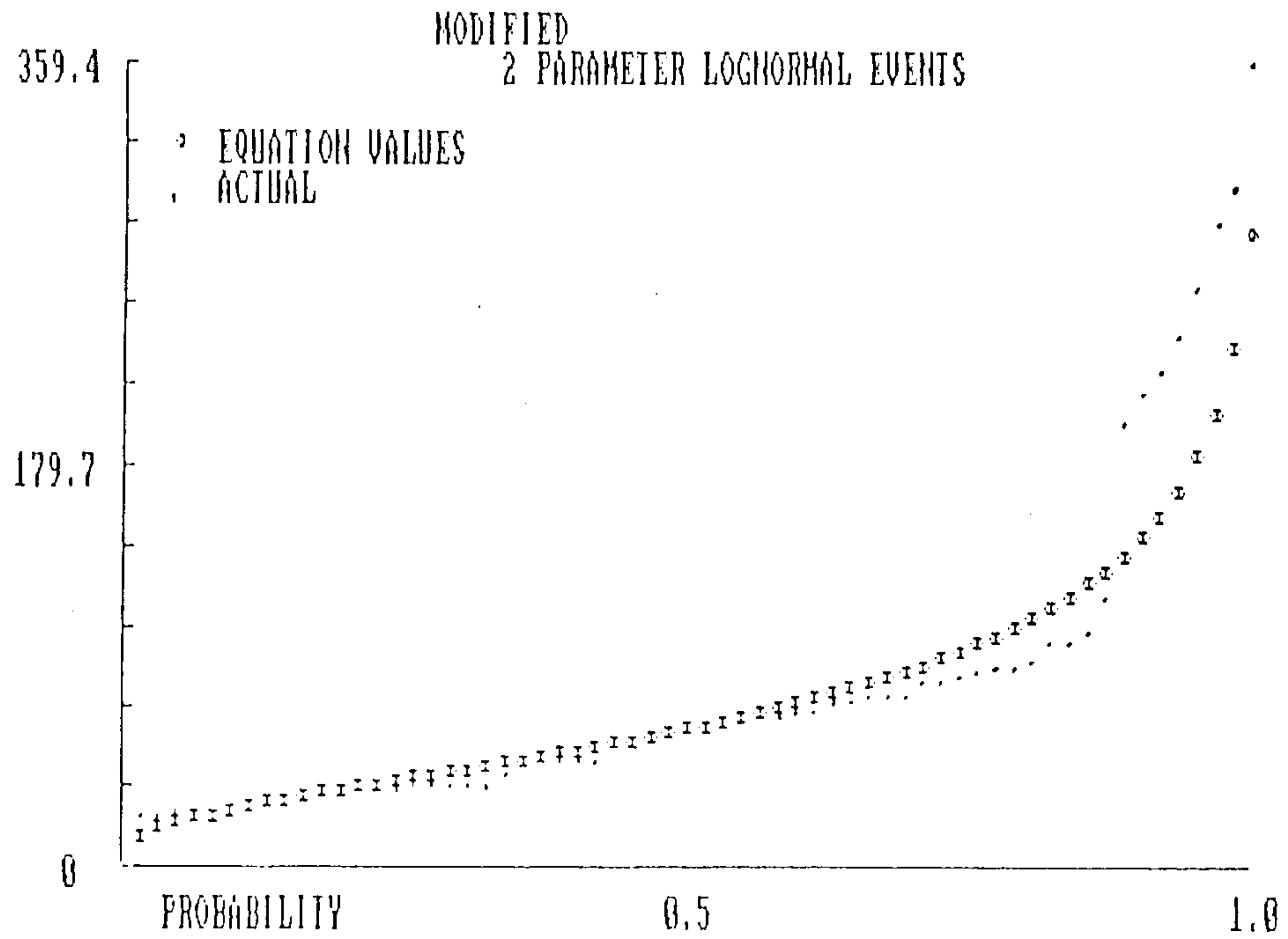

Figure 30. Log-Normal Distribution - Modified - Detention Time Distribution.

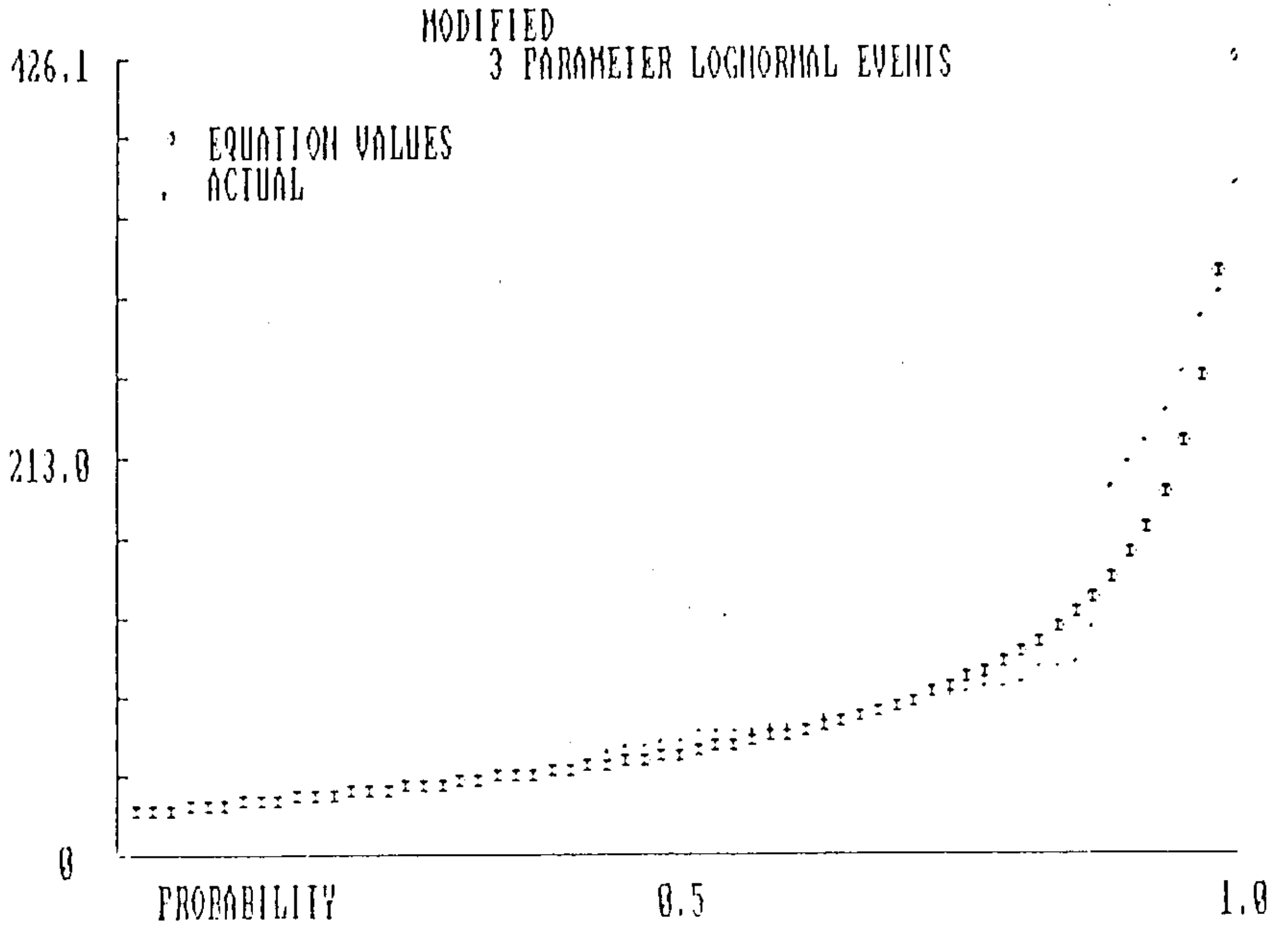

Figure 31. 3-Parameter Log-Normal - Modified - Detention Time Distribution.

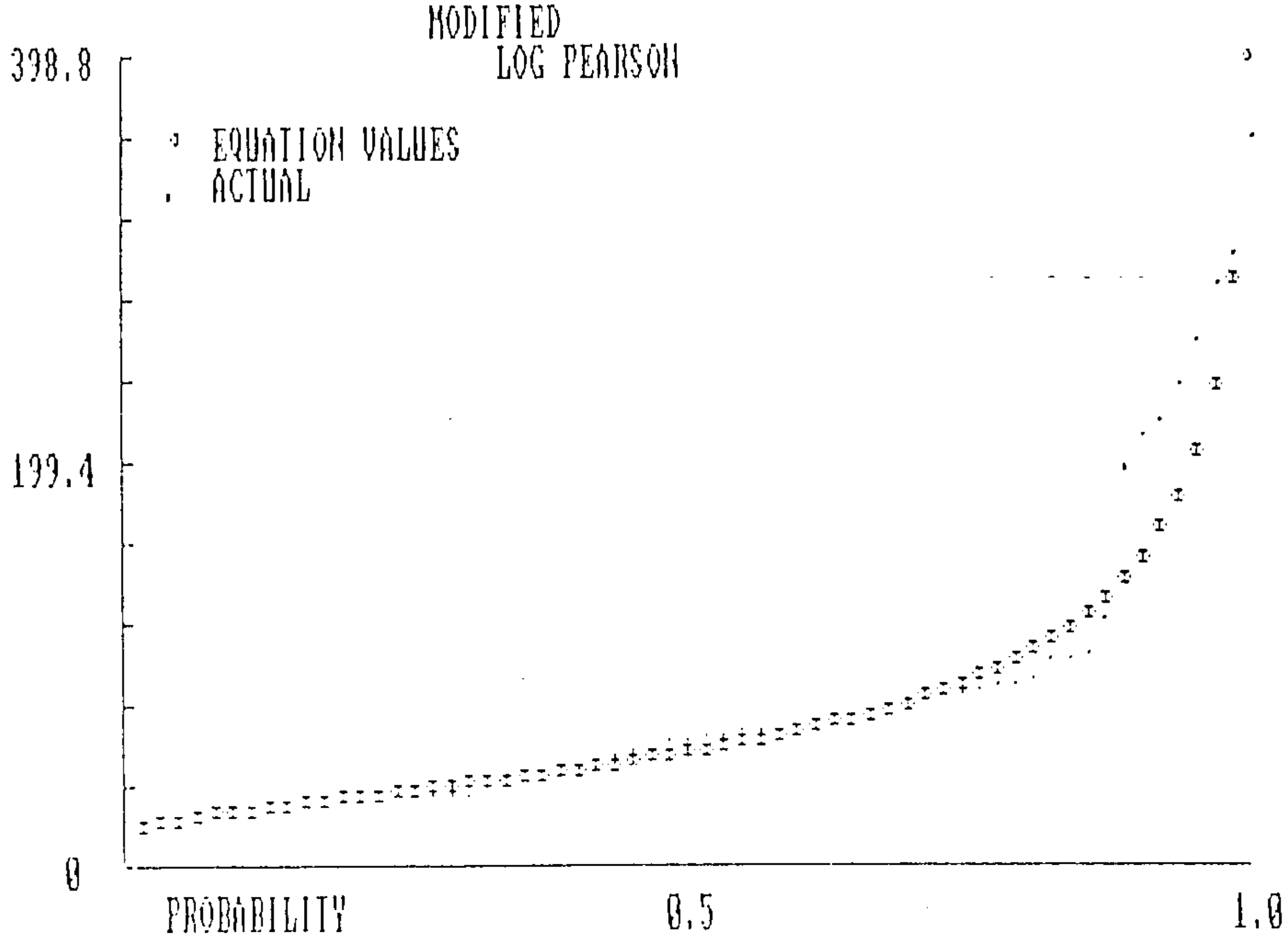

Figure 32. Log-Pearson Type III - Modified - Detention Time Distribution.

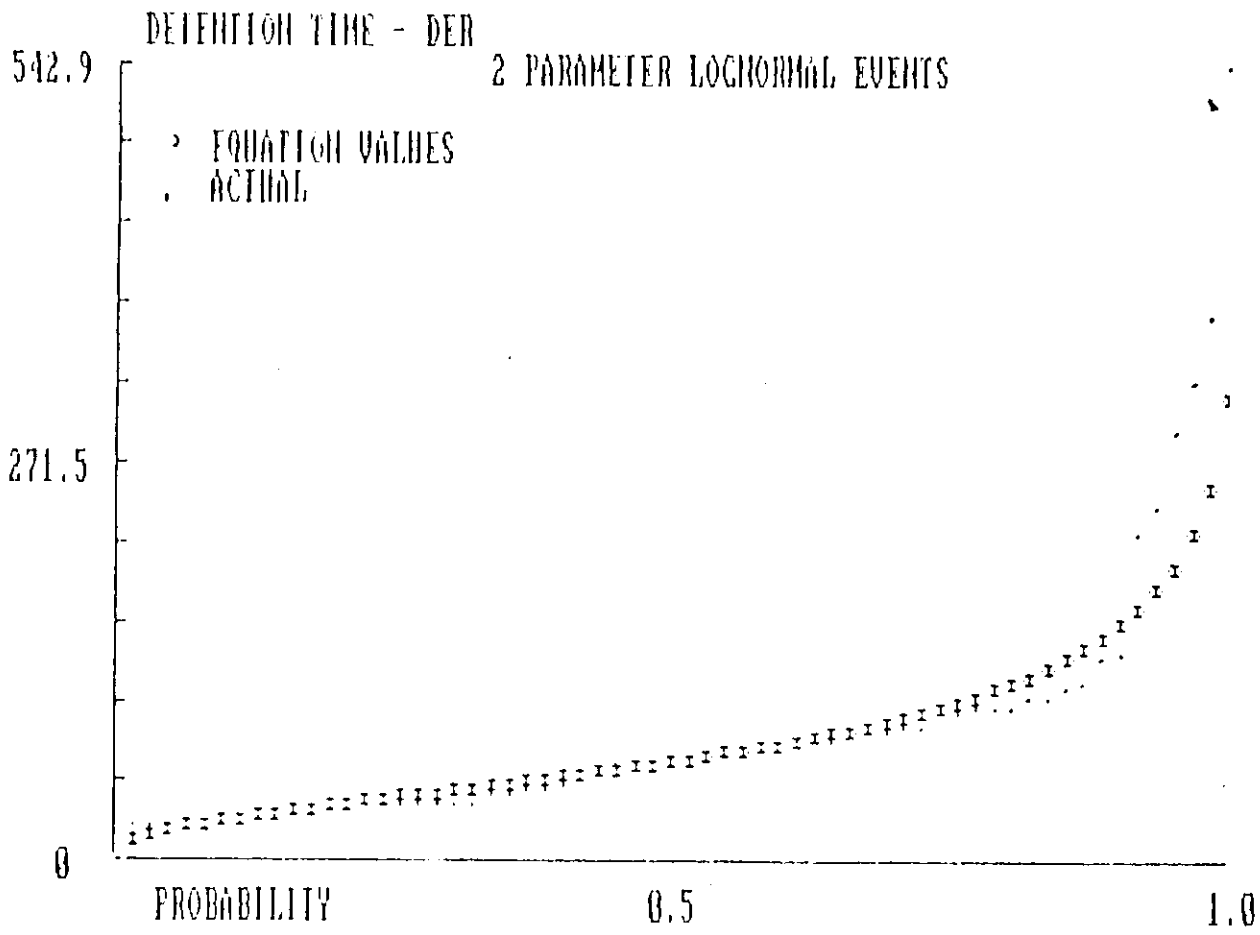

Figure 33. Log-Normal - DER - Detention Time Distribution.

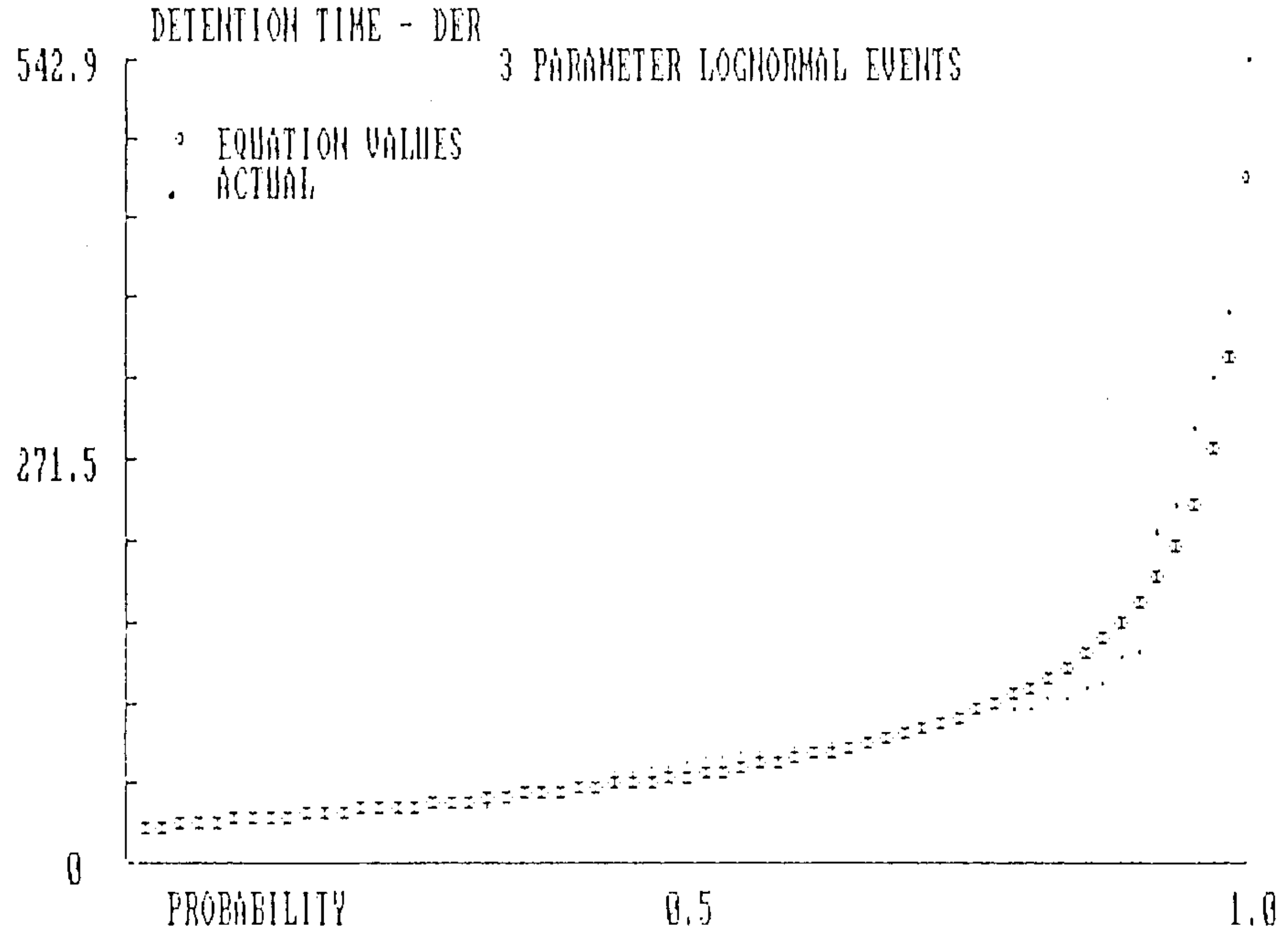

Figure 34. 3-Parameter Log-Normal - DER - Detention Time Distribution.

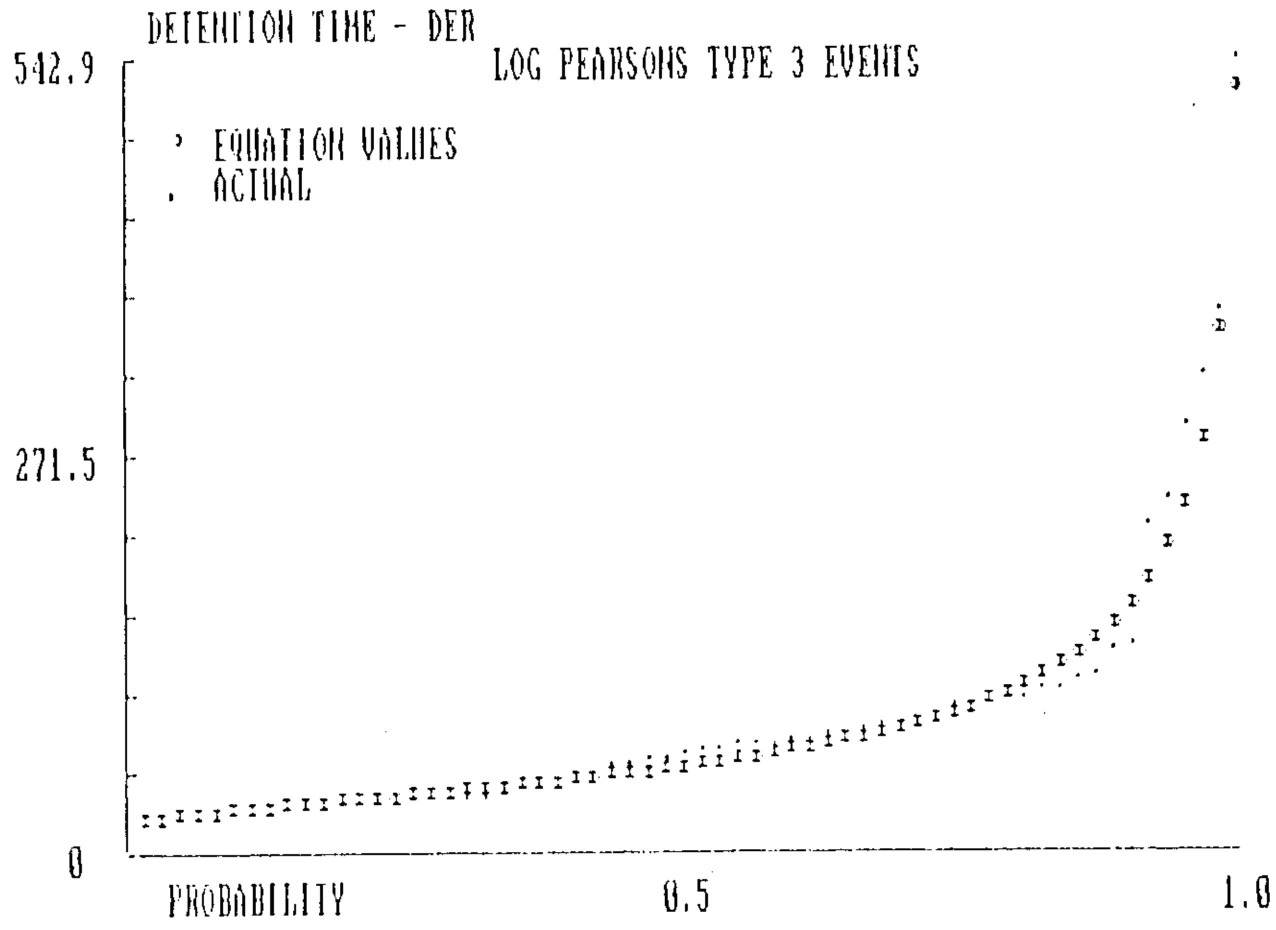

Figure 35. Log-Pearson Type III - DER - Detention Time Distribution.

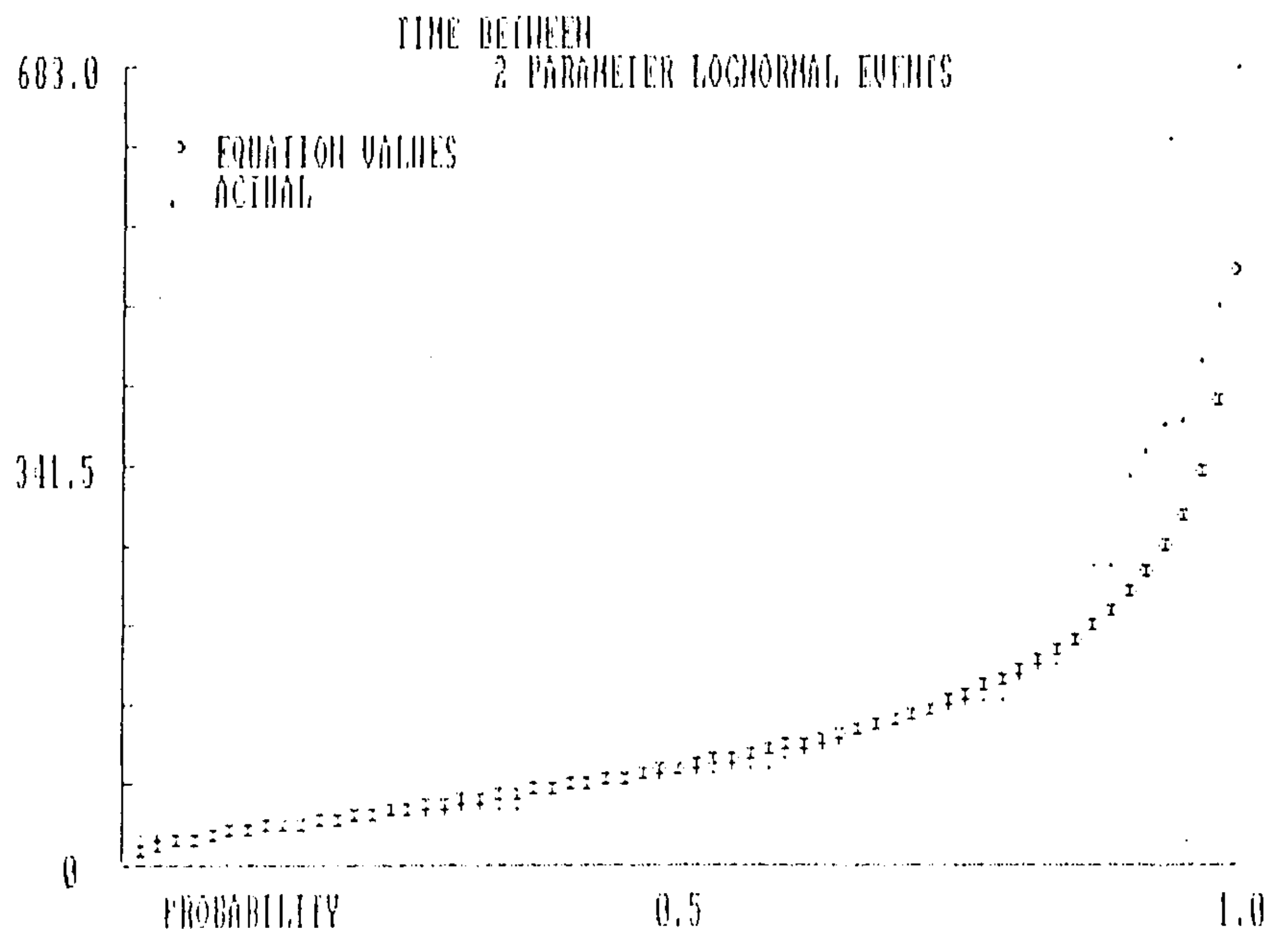

Figure 36. Log Normal - Time Between Detention Times Distribution.

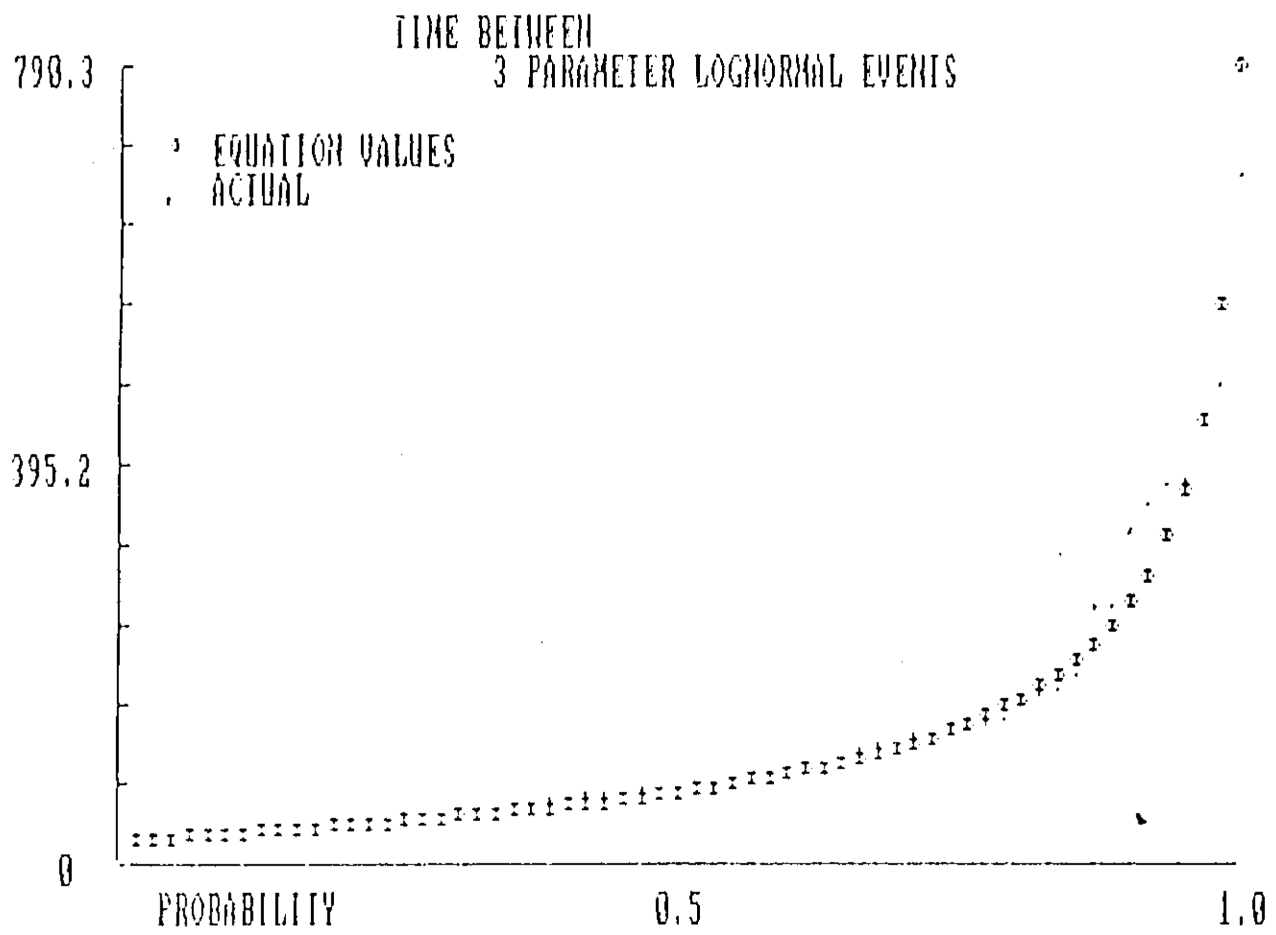

Figure 37. 3-Parameter Log-Normal - Time Between Detention Times Distribution.

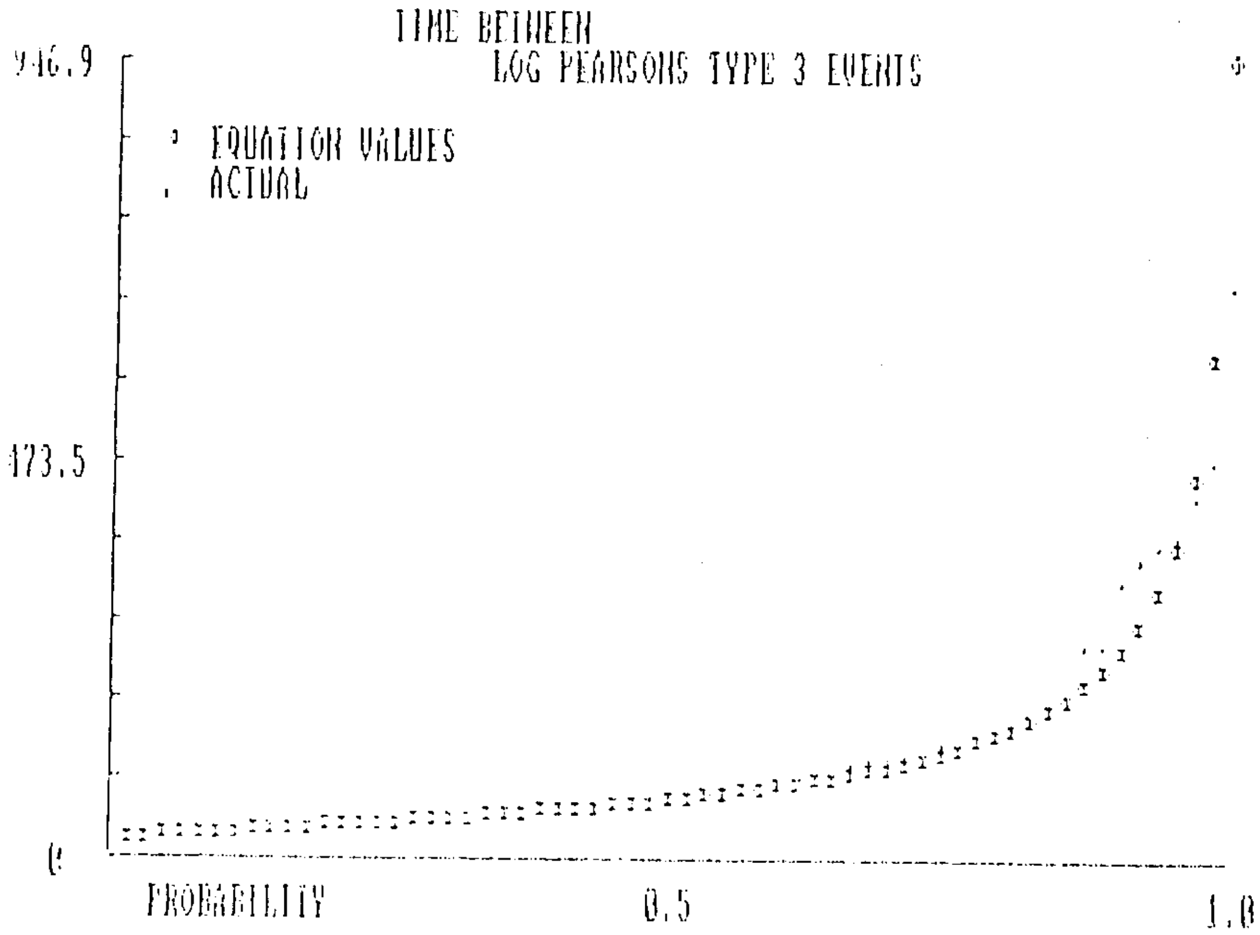

Figure 38. Log-Pearson Type III - Time Between Detention Times Distribution.

APPENDIX C

POND AND GROUNDWATER ELEVATION DATA AT COMMERCIAL SITE USE FOR CALCUALTION OF BANK INFILTRATION RATES

TABLE C.1

POND ELEVATION DATA
COMMERCIAL SITE
EXPERIMENT #1
4/29/87
Groundwater Elevation

DAY	TIME	SOUTH (feet)	NORTH (feet)	POND (feet)	INFILTRATION RATE (ft/hr)
April 29	15:10	64.58	63.25	66.70	----
	15:55	64.62	63.42	66.50	0.260
	17:45	64.62	63.50	66.34	0.087
	19:45	64.62	63.58	66.25	0.045
April 30	00:00	64.62	63.58	66.12	0.029
	08:00	64.62	63.58	66.04	0.010
	12:00	64.58	63.58	66.00	0.010
	20:00	64.58	63.58	65.92	0.010
May 1	08:00	64.58	63.50	65.88	0.003

Notes: April 30 was a cloudy day, however some evaporation can be expected.

TABLE C.2
POND ELEVATION DATA
COMMERCIAL SITE
EXPERIMENT #2
9/18/87
Groundwater Elevation

DAY	TIME	(feet)	(feet)	(feet)	RATE (ft/hr)
Sept 18	08:50	64.62	63.75	66.00	----
	09:50	64.62	63.82	65.75	0.250
	11:00	64.62	63.87	65.62	0.111
	11:50	64.62	63.89	65.58	0.048
	15:30	64.62	63.84	65.46	0.033
	17:45	64.62	63.79	65.42	0.018

TABLE C.3
POND ELEVATION DATA
COMMERCIAL SITE
EXPERIMENT #3
9/25/87
Groundwater Elevation

DAY	TIME	SOUTH (feet)	NORTH (feet)	POND (feet)	INFILTRATION RATE (ft/hr)
Sept 25	09:00	64.62	63.79	65.50	----
	09:30	64.62	63.79	65.38	0.240
	10:30	64.62	63.81	65.25	0.130
	13:25	64.62	63.75	65.17	0.045
	17:00	64.58	63.54	65.08	0.022
Sept 26	00:00	64.58	63.50	64.96	0.017
	08:00	64.58	63.42	64.88	0.010
	14:00	64.50	63.37	64.84	----*

* Could be affected by evaporation.

TABLE C.4
POND ELEVATION DATA
COMMERCIAL SITE
EXPERIMENT #4
9/29/87
Groundwater Elevation

DAY	TIME	POND (feet)	INFILTRATION RATE (ft/hr)
Sept 29	15:15	66.18	----
	16:00	66.08	0.133
	17:00	66.00	0.080
	19:00	65.86	0.070
	20:00	65.80	0.060
	22:00	65.69	0.055
Sept 30	00:00	65.62	0.035
	02:00	65.58	0.020
	04:00	65.54	0.020
	08:10	65.50	0.010

NOTE: Groundwater table elevation (North side) averaged 64.25 feet and varied only 0.04 feet during sampling.

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