Report No. 248

# EVALUATION OF DETENTION BASIN PERFORMANCE IN THE PIEDMONT REGION OF NORTH CAROLINA

by

Jy S. Wu, Ph.D., P.E.
Associate Professor

Department of Civil Engineering
University of North Carolina at Charlotte
Charlotte, North Carolina 28223

July 1989

Water Resources Research Institute
OF THE UNIVERSITY OF NORTH CAROLINA

UNC-WRRI-89-248

# EVALUATION OF DETENTION BASIN PERFORMANCE IN THE PIEDMONT REGION OF NORTH CAROLINA

By

Jy S. Wu, Ph.D, P.E.
Associate Professor

Department of Civil Engineering
The University of North Carolina at Charlotte
Charlotte, North Carolina 28223

The work upon which this publication is based was supported in part by funds provided by the Water Resources Research Institute of The University of North Carolina.

Project No. 70077

July 1989

## ACKNOWLEDGEMENTS

This project was funded in part by The Water Resources Research Institute of The University of North Carolina. Additional funding was made available through the Engineering Major Grants program from The University of North Carolina at Charlotte.

The author would like to acknowledge the assistance and encouragement provided by Drs. David Moreau and James Stewart of the Water Resources Research Institute; Mr. Bill Kreutzberger, Ms. Beth McGee, Dr. Bob Holman, and Mr. John Dorney of the Water Quality Planning Branch, Division of Environmental Management, N.C. Department of Natural Resources and Community Development; and Mr. Edward Holland of the Triangle J. Council of Government. Mr. Harold Eddins and his staff and Mr. Kent Crawford from the U.S. Geological Survey assisted in field instrumentation and stream gage data processing. The use of automatic samplers was made available by the N.C. Water Quality Laboratory. The Engineering Department of the City of Charlotte helped identify potential study sites. Numerous undergraduate and graduate civil engineering students participated in the field sampling program and data analysis, particularly Bill Langley, Fred Wu and Tony Yang. Cooperation provided by property owners of the study site is greatly appreciated.

## DISCLAIMER STATEMENT

# ABSTRACT

Recent concerns on controlling urban runoff pollution have resulted in nationwide investigations on the beneficial use of existing and potential detention facilities. Particularly, the use of wet detention ponds has been considered a best management practice for reducing nonpoint urban pollutants to receiving streams. Detention ponds which are properly designed can serve not only for flood control but also for the retention of sediment and other pollutants associated with settleable particulates. A data base for assessing the performance of such dual-function detention ponds is critically needed.

This report summarizes results of a stormwater sampling program conducted on three existing urban wet detention ponds in the Piedmont region of North Carolina in the City of Charlotte. These ponds were not originally designed for water quality control. A total of eleven storm events was monitored. Runoff samples were analyzed for total suspended solids, total and ortho phosphorus, total Kjeldahl and ammonia nitrogen, and metals of iron, zinc, copper and lead. The removal efficiency of pollutants for each detention pond was computed as the percent difference of the total pollutant mass entering and leaving the detention pond. An U.S.EPA model was employ to derive a relationship for estimating the size of detention ponds to achieve targeted levels of water quality improvement. It is estimated that about 1.0% to 2.0% of the watershed area would be needed for siting of wet detention ponds to accomplish a sediment removal efficiency of 70% or better. The quality of storm runoff from the Piedmont North Carolina urban areas was found to be generally better than that reported by the National Urban Runoff Program.

The research findings are consistent with the North Carolina State Government guidelines for sizing wet detention ponds for water quality considerations. Results of this study are based on Piedmont watershed characteristics and should not be applied to other regions without consideration of site specific information.

## TABLE OF CONTENTS

Page No.

## LIST OF FIGURES

# LIST OF TABLES

## SUMMARY AND CONCLUSIONS

The degree of urban runoff contamination is highly variable and site specific, depending on the characteristics of the drainage area as well as rainfall intensity and duration. The State of North Carolina, through the Department of Natural Resources and Community Development, has developed a stormwater quality management program to assist local government and developers in implementing best management practices (BMPs), including wet detention ponds, for protection of water supply watersheds. Pollutant removal targets of 62% and 85% for total suspended solids were chosen for storm events associated with the first 1/2-inch and 1-inch of rainfall, respectively. The use of wet detention ponds or other stormwater treatment systems is required by regulation in some cases for coastal counties and encouraged in water supply watersheds. Information on the beneficial use of detention ponds in the Piedmont of North Carolina is limited. Consequently, a field monitoring project was implemented to collect stormwater quantity and quality data from three existing urban wet detention ponds in the City of Charlotte. The project provides a data base to examine the performance of wet detention ponds in the Piedmont. The relationship of pond size and water quality improvement was examined and characterized by presentation of pollutant removal data.

Results of this investigation reveal that the water quality benefits of Piedmont North Carolina wet ponds can be correlated to a surface area ratio and is predictable by an EPA computer model. The surface area ratio or SA/DA is defined as the ratio of the surface area of a detention pond to its contributing drainage area (pond surface area:contributing drainage area). The Piedmont wet detention ponds were found to be effective in removing total suspended solids (TSS) load in urban runoff. The SA/DA ratios required to achieve 70% or better TSS removal would be 1% or greater. In order to provide 80% or better TSS removal, the SA/DA ratios would need to increase to 2% or greater.

The North Carolina design guidelines recommend SA/DA ratios ranged from 0.5-1% or 1.3-5.7% for achieving 62% or 85% TSS removal, respectively, depending on basin depths, watershed characteristics and protection requirements. In comparison, the SA/DA ratios obtained from this study are 1% or 4.5% for achieving the targeted TSS removal of 62% or 85%, respectively. The average depths of detention ponds investigated ranged from 4 to 8 feet, whereas the average depths cited by North Carolina guidelines are 3 to 6 feet. The rate of sediment accumulation in Piedmont wet detention ponds had not been studied; however, research conducted elsewhere suggests a 13% reduction of the storage capacity over a 10-year period.

The removal efficiencies for other pollutants at SA/DA ratios of 1-2% are: iron (60%), zinc (40%), TKN (30%), and TP (45%). The nutrient level in one of the detention ponds was found to be affected by the presence of urban geese. The contribution of TKN and TP by urban geese accounted for 5% and 7% of the total input, respectively. Lead and copper were not detectable in storm samples or in pond effluent by flame atomic absorption spectrophotometry technique. The pollutant removal reported should be viewed as long-term average performance of Piedmont North Carolina wet detention ponds.

The quality of storm runoff from the study area was characterized by an event mean concentration (EMC) and a particle size distribution. The EMCs are 135 mg/l for total suspended solids, 0.88 mg/l for total Kjeldahl nitrogen, 0.22 mg/l for ammonia nitrogen, 0.14 mg/l for total phosphorus, 0.10 mg/l for ortho-phosphorus, 6.11 mg/l for total iron, 0.24 mg/l for soluble iron, 66 ug/l for total zinc, and 20 ug/l for soluble zinc. The average concentrations of total suspended solids from the detention pond effluent vary from 10 to 50 mg/l, corresponding to 63-93% reduction of the influent sediment concentration (135 mg/l). The reduction of EMCs for Zn and Fe are 50-75% and 40-85%, respectively. There is negligible reduction of EMCs for total Kjeldahl nitrogen and total phosphorus.

Phosphorus in storm runoff appears predominantly as soluble phosphorus; its removal mechanism is dictated by the dilution capacity of the detention ponds. On the other hand, the removal of particulate pollutants is achieved by sedimentation. Fine particles require a longer detention time to settle. The mean particle size (50 percentile) in storm runoff is 6 microns (equivalent to a settling velocity of 0.3 ft/hr) corresponding to the very fine silt and clay soils in the Piedmont. The particle sizes reported are viewed as the long-term average distribution because of variations in wash-off mechanism at the various stages of a storm event and among different storms.

The degree to which these Piedmont wet detention ponds reduce hydrographic peaks was found to be less than satisfactory because significant runoff enters near the downstream portion of the detention ponds. This is attributed to the extensive development of housing projects surrounding the detention ponds.

In summary, the following conclusions can be made.

(1) Although the existing Piedmont urban wet detention ponds were not originally built for the purpose of water quality improvement, they are found to be effective for reducing urban runoff pollution.

(2) The quality of storm runoff from the study area is better than that reported by the National Urban Runoff Program. The study site is representative of Piedmont urban

settings where watershed management and/or maintenance activities are emphasized to reduce nonpoint source pollution.

(3) A relationship between pollutant removal effectiveness and the surface area ratio of detention ponds has been obtained to characterize the water quality improvement performance of existing wet detention ponds in the Piedmont region of North Carolina.

(4) A computer model developed by U.S. EPA has been found to be reasonably useful for initial sizing of urban wet detention ponds to achieve targeted levels of pollution control. If local meteorological, hydrological and soil properties are available, the model could provide an estimation of the long-term efficiency of sediment control using wet detention ponds.

(5) Results of this study are based on Piedmont watershed characteristics and should not be applied to other regions without consideration of site specific information.

# INTRODUCTION

In the late seventies, a series of Best Management Practices (BMPs) were developed to control nonpoint pollution due to urban runoff. Urban wet detention ponds are considered as one of the BMPs capable of improving the quality of runoff from urban areas. With proper design, urban wet detention ponds can serve not only for flood control but also for retention of sediment and other pollutants associated with settleable particulates. They also provide aesthetic amenities and recreational opportunities to the community. The use of wet detention ponds for controlling urban runoff pollution is required by regulation in some cases in the North Carolina coastal counties and encouraged in water supply watersheds. Information on the beneficial use of these multi-purpose detention ponds in the Piedmont region of North Carolina is limited. There is a need to obtain field performance data from existing wet detention ponds for initial sizing based on water quality improvement. The relationship between pollutant removal efficiency and pond sizes needs to be developed.

Consequently, a field sampling program (1987-1988) was initiated to establish a data base to examine the performance of existing urban wet detention ponds in the Piedmont region of North Carolina. Based on results obtained from field monitoring and computer modeling, a relationship was obtained to evaluate the effectiveness of wet detention ponds in controlling urban runoff pollution.

# LITERATURE REVIEW

Recent concerns about water quality problems related to nonpoint source pollution have resulted in a nationwide investigation of the beneficial use of existing and potential detention facilities. A number of remedial measures referred to as "best management practices" (BMPs) have been described by the Metropolitan Washington Council of Government (Schueler, 1987). These include the use of wet and extended detention ponds, infiltration trenches, infiltration basins, porous pavements, water quality inlets and vegetative systems. Among others, wet detention ponds also referred to as wet ponds have been considered a promising BMP for controlling urban runoff pollution. For instance, installing wet ponds at strategic locations within a watershed could eliminate a major portion of nonpoint pollutant loadings for the entire area (DEM, 1985). In contrast, dry ponds have been found inefficient in removing suspended solids and other pollutants. Often, re-suspension of deposited sediment in dry ponds results in negative removal efficiencies (Dally, 1983).

Studies on controlling runoff pollution in Chicago reveal that from 91 to 95% removal of suspended sediment and from 76 to 94% removal of copper, iron, lead, and zinc could be achieved by an urban lake. The accumulated sediment in the lake appears in the form of organic-rich mud, with an accumulation rate of 20 millimeters per year, which is equivalent to a 13 percent loss in lake storage over a 10-year period (Striegl, 1987).

The National Urban Runoff Program (NURP) implemented a number of projects to study the performance of nine wet ponds (U.S. EPA, 1983). The physical setting of these ponds varied widely including oversized section of storm drains installed below street level, ponds or small lakes on streams, flood control basins, a farm pond, and a golf course pond. The surface area ratios of these ponds ranged from 0.01 to 2.85%. Surface area ratio, or SA/DA, is defined as the ratio of pond surface area to contributing drainage area. Results of the NURP studies indicated that when wet ponds are adequately sized, particulate removals in excess of 90% (TSS, lead) can be obtained. Pollutants with significant soluble fractions in urban runoff show lower reductions; on the order of 65% for total phosphorus and approximately 50% for BOD, COD, TKN, copper and zinc. A probabilistic model was subsequently developed for planning level evaluations and design decisions on the use of wet ponds and recharge basins (U.S. EPA, 1986). Typically, the performance of wet ponds relating to SA/DA can be illustrated in Figure 1.

Methods are available to predict sediment trapping efficiencies in detention ponds (Camp, 1945; Rausch and Heineman, 1975; Griffin et al, 1980; McCuen, 1980; and Wu and Ahlert, 1985). A study on hydrodynamics and sedimentation in detention ponds concluded that (a) ponds constructed with length-to-width ratio of 2-to-1 or greater provide a better trapping efficiency of sediment; (b) shallow and long ponds are better than deep and short ponds--a minimum depth of 3 feet is recommended; (c) poor performance of dry ponds is attributed, in part, to the fluctuation of flow-through velocities during a storm event, permitting re-suspension of deposited sediment; and (d) performance of wet ponds depends greatly on influent particle size distributions (Wu and Ahlert, 1986). Issues of hydraulic function, public safety, maintenance, water quality and aesthetic aspects of outlet controls for detention ponds have also been investigated (ASCE, 1985).

The preferred method of water supply protection in North Carolina is based on the concept of low density development and is summarized in Figures 2 and 3 (Harrell, 1988). Structural controls for runoff are not required if the impervious cover is less than 6% inside the critical area or less than 12% in areas outside the critical area which are served by septic tanks. The critical area is the area within 1/2 to 1 mile of the reservoir or intake point depending on the watershed size. In urbanized areas or where sewer lines are needed, the impervious cover can go up to 30% outside the critical area. However, when impervious

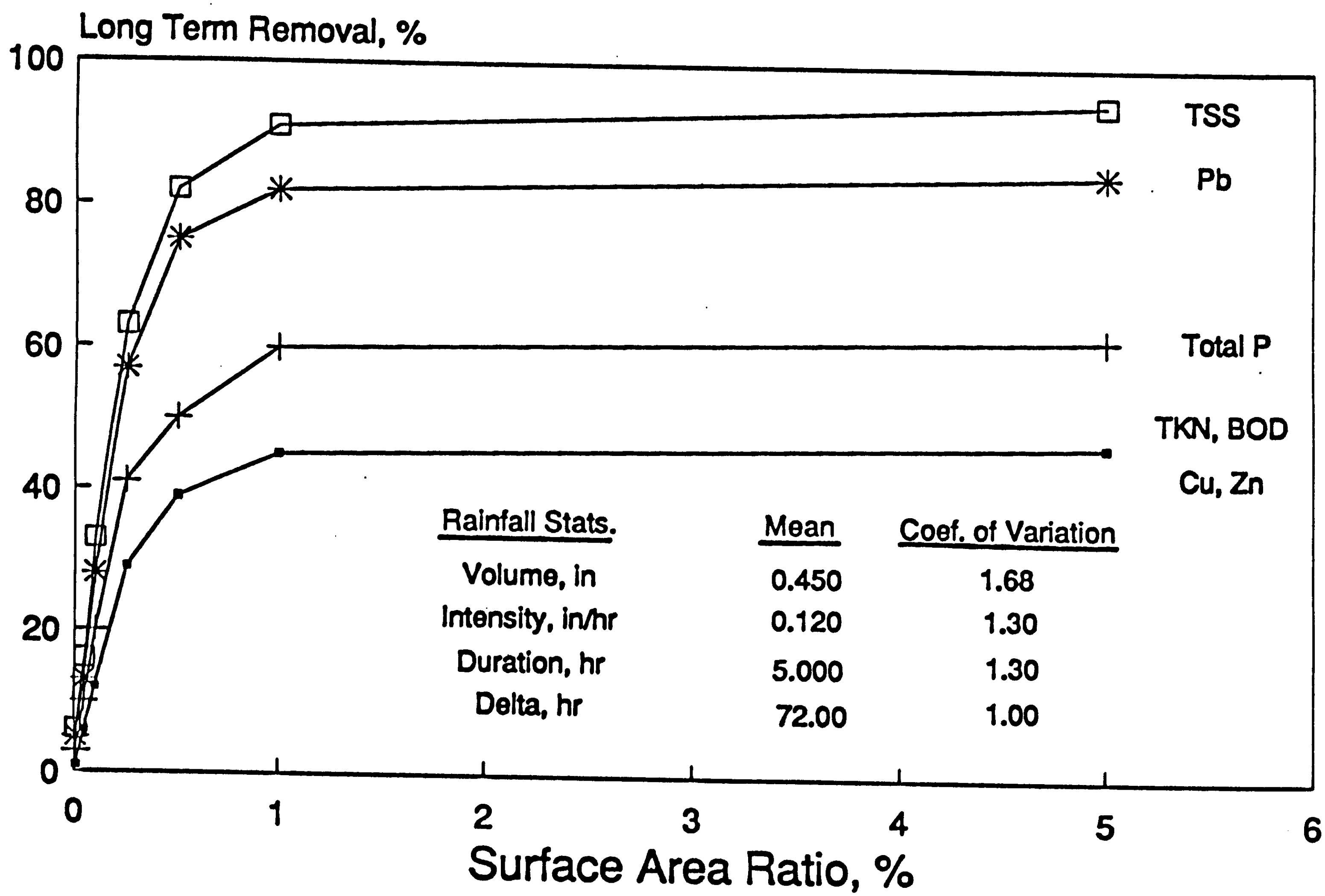


**Figure 1. Performace of Wet Detention Ponds, Southeast USA (EPA, 1986)**

Impervious %
≤ 6% ?

NO

**TO BE USED IN UNAVOIDABLE SITUATIONS**
**Wet Detention Basin**

SA/DA % for basin depths

| Impervious (%) | 3.0ft | 3.5ft | 4.0ft | 5.0ft | 6.0ft |
|---|---|---|---|---|---|
| 7-29 | 1.8 | 1.5 | 1.4 | 1.1 | 1.0 |
| 30 | 2.4 | 2.1 | 1.8 | 1.5 | 1.3 |
| 50 | 4.0 | 3.5 | 3.0 | 2.5 | 2.1 |
| 70 (max) | 5.7 | 4.8 | 4.3 | 3.5 | 2.9 |

(controls 1-inch rainfall)
Interpolate intermediate values
* Surface area basin/drainage area * 100

YES

**PREFERRED METHOD**
No structural controls needed

Figure 2. Design Guidelines for Wet Detention Ponds in Piedmont, North Carolina (Inside Critical Area)

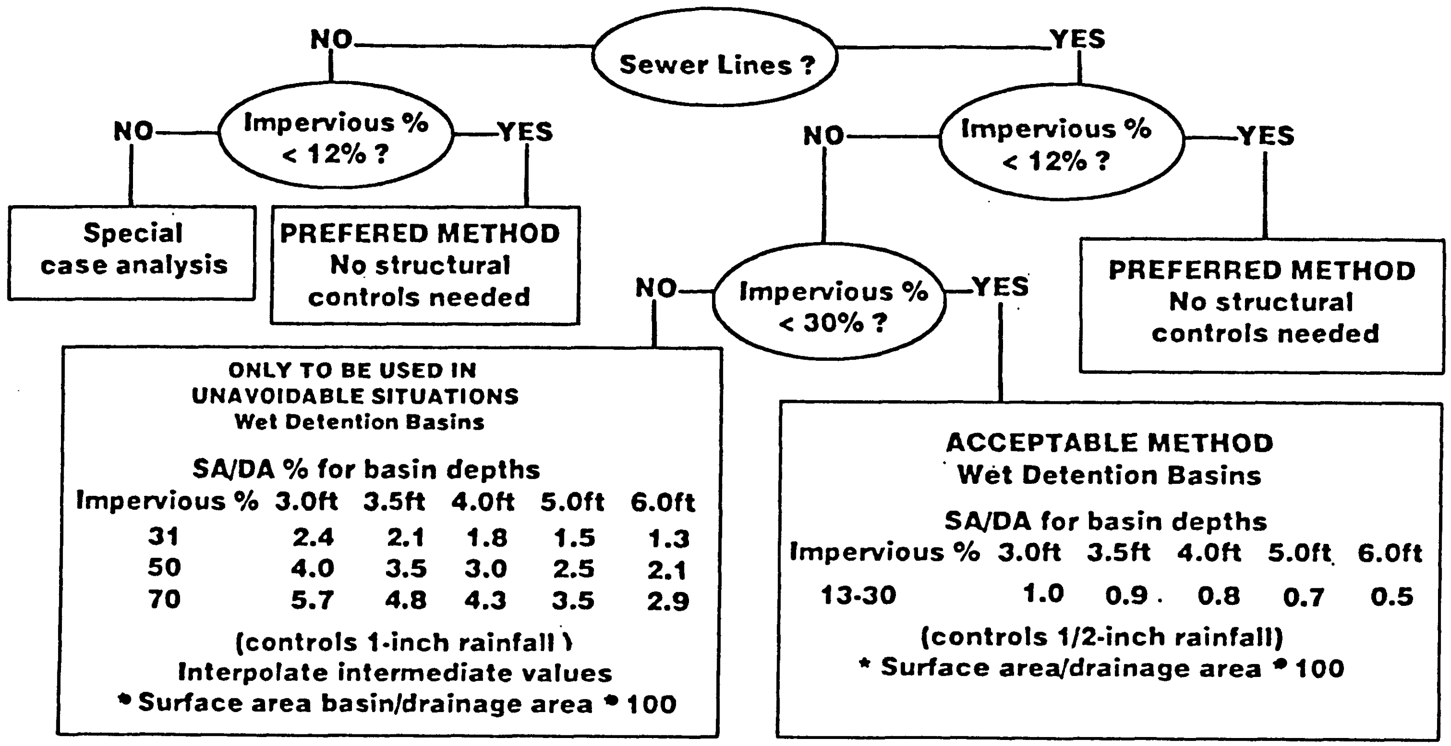


**Figure 3. Design Guidelines for Wet Detention Ponds in Piedmont, North Carolina (Outside Critical Area)**

cover exceeds 6% inside the critical area, structural controls are required to capture runoff generated by the first 1-inch of rainfall. For areas outside the critical area which are served by sewer lines, structural controls are required to retain runoff generated by the first 1/2-inch of rainfall when impervious cover is 13-30%, or to retain runoff generated by the first 1-inch of rainfall when impervious cover exceeds 30%. Figures 2 and 3 also present the required SA/DA ratios which may satisfy targeted TSS removal criteria applicable to the Piedmont region of North Carolina.

# RESEARCH METHODOLOGY

## Characteristics of Study Sites

A reconnaissance of detention ponds in the Charlotte-Mecklenburg area was conducted during the summer of 1986 to identify potential study sites. A large number of drainage plans, provided by the Engineering Department of the City of Charlotte, were reviewed. After extensive field visits of approximately 15 sites, three urban wet detention ponds were selected for monitoring; they are Lakeside (LS), Waterford (WF) and Runaway Bay (RB) ponds. These ponds are located near Monroe Road in eastern Charlotte in the Piedmont region of North Carolina. The entire watershed has a drainage area of 437 acres and comprises three subareas of Lakeside (65 acres), Waterford (302 acres), and Runaway Bay (70 acres). The detention ponds, which were built in the late seventies or early eighties, are located on the respective subareas. None is designed as a water quality control device. The watershed layout and information pertinent to each detention pond are given in Table 1 and Figures 4-8. A pictorial view of the study area is presented in Figures 9-12.

The upper portions of Lakeside and Waterford subareas consist primarily of single family residential land use. Adjacent areas surrounding LS and RB ponds are characterized by intensive development of multi-family housing such as condominiums and apartments. Storm runoff generated from the adjacent impervious areas drains directly into the ponds either as overland flow or via a number of storm pipes. In general, each detention pond receives flows from its upstream channel (upstream inflow) and storm runoff from adjacent impervious areas (local runoff), and discharges through a weir structure (downstream outflow).

The relative size of a detention pond can be expressed by its surface area ratio (SA/DA). These ratios are calculated based on pond surface and drainage subarea acreages. Thus, SA/DA ratios for LS, WF and RB ponds are 7.5%, 0.6% and 4.6%, respectively. RB pond is located downstream of LS and WF ponds and receives their combined outflows. Consequently, the SA/DS ratio for RB pond, with respect to the total drainage area, is 0.75%. The overall ratio for all three ponds and the entire watershed is 2.27%. The above area ratios were correlated to pollutant removal efficiency.

## Installation of Gaging Stations

A total of five stream flow gaging stations and one rain gage, all constructed and operated according to U.S. Geological Survey guidelines, were installed in the field (Figures

**Table 1. Watershed and Detention Pond Characteristics**

| | Lakeside | Waterford | Runaway Bay |
|---|---|---|---|
| | Watershed Characteristics | | |
| Land Use | Single Family<br>Apartment<br>Condominium | Single Family<br>Apartment<br>Wooded | Apartment<br>Wooded |
| Acreage, acres | | | |
| (1) | 30 | 302 | 367 |
| (2) | 35 | -- | 70 |
| Total | 65 | 302 | 437 |
| Imperviousness,% | | | |
| (1) | 25 | 30 | 33 |
| (2) | 65 | -- | 65 |
| Average | 46 | 30 | 38 |
| | Detention Pond Characteristics | | |
| Acreage, acres | 4.90 | 1.80 | 3.30 |
| Volume, acre-ft | 38.80 | 5.10 | 12.30 |
| Mean Depth, ft | 7.90 | 2.80 | 3.80 |
| SA/DA,% | 7.50 | 0.60 | 2.30 |

(1) Measured above the upstream inflow channel
(2) Including surrounding area of the detention pond
Note: Imperviousness was estimated at 25% and 30% for 1/2-acre and 1/3-acre residential houses, and 65% for town houses.

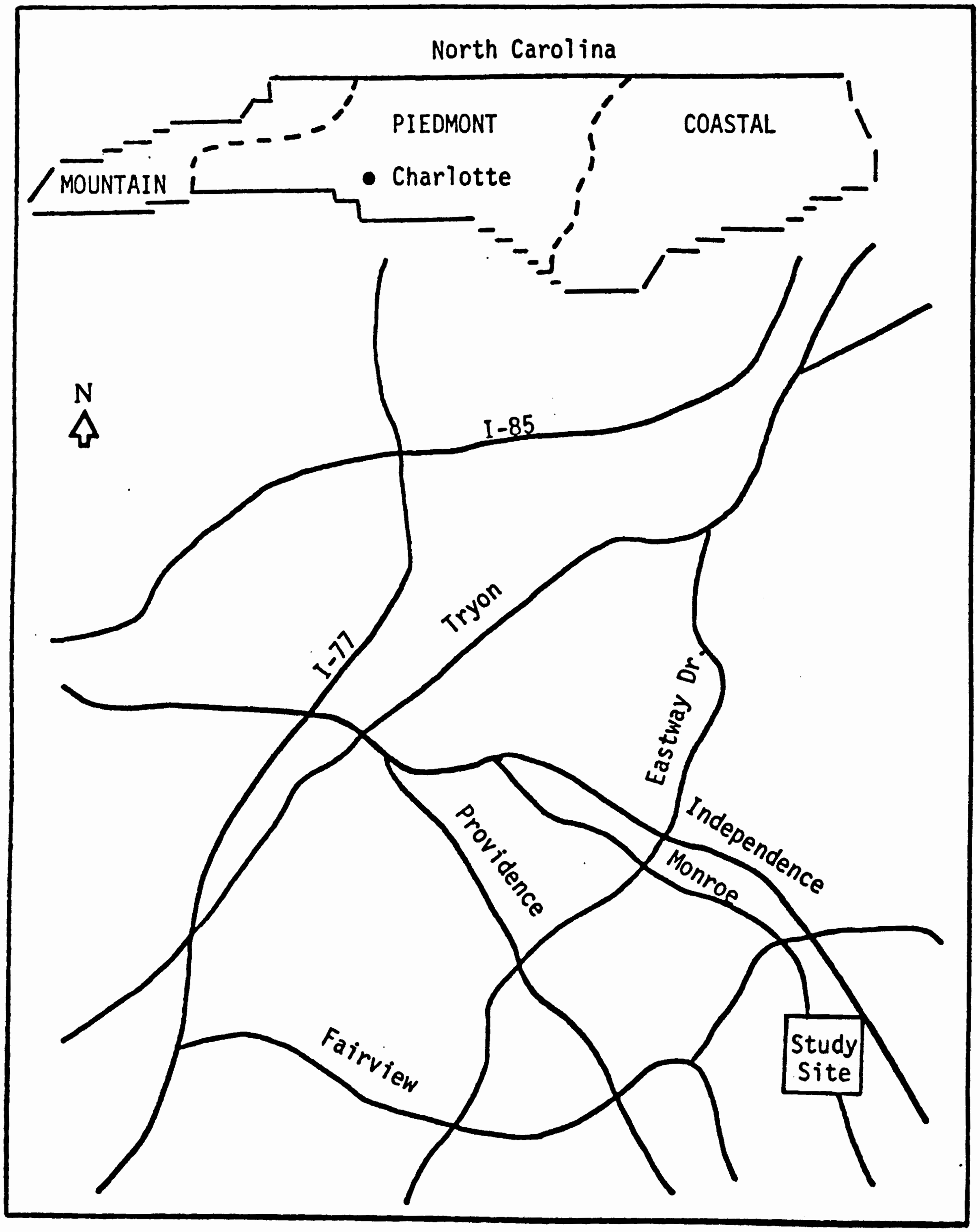


**Figure 4. Location of the Study Site in Charlotte, North Carolina**

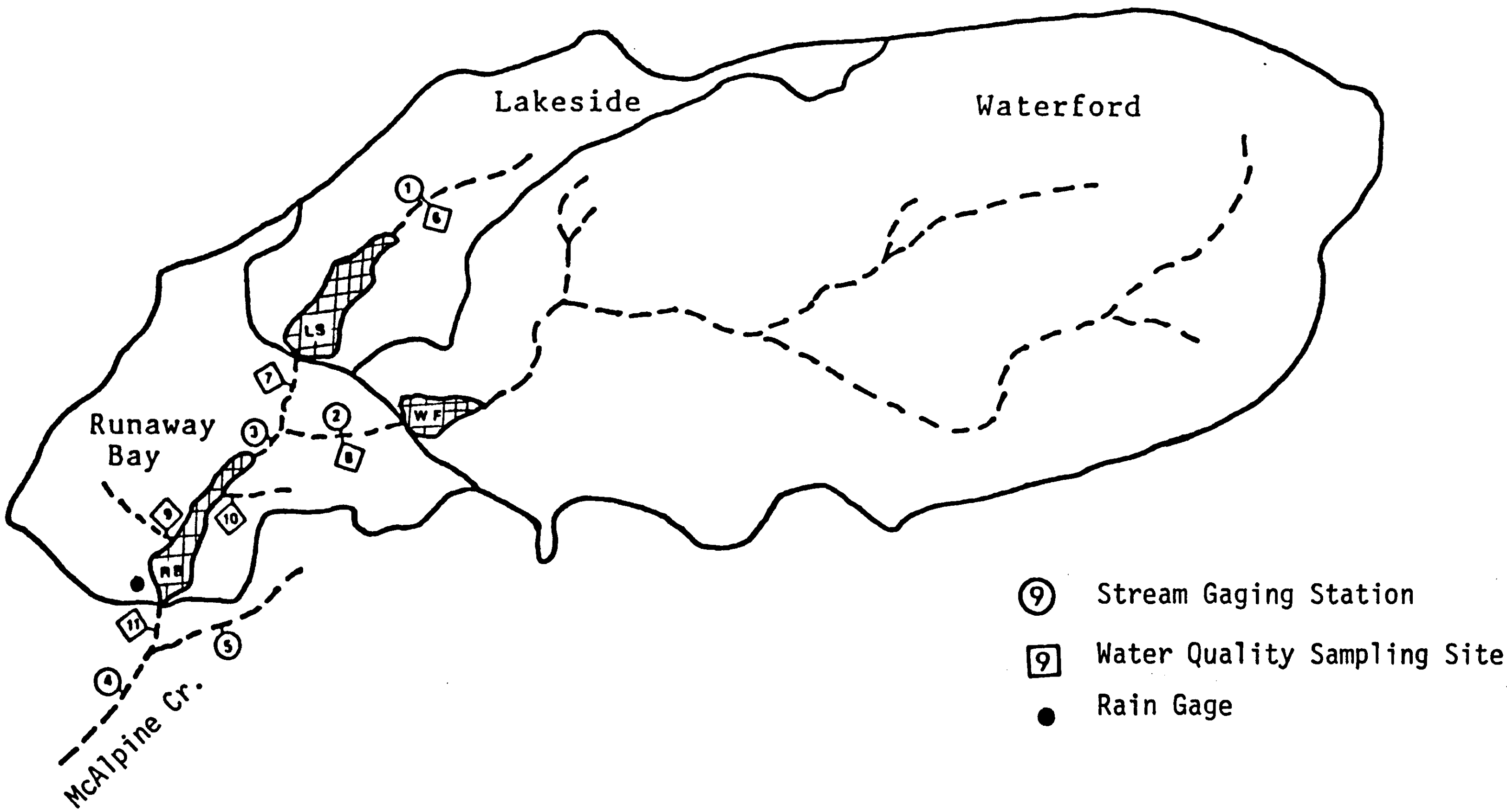


**Figure 5. Watershed Layout and Monitoring Stations**

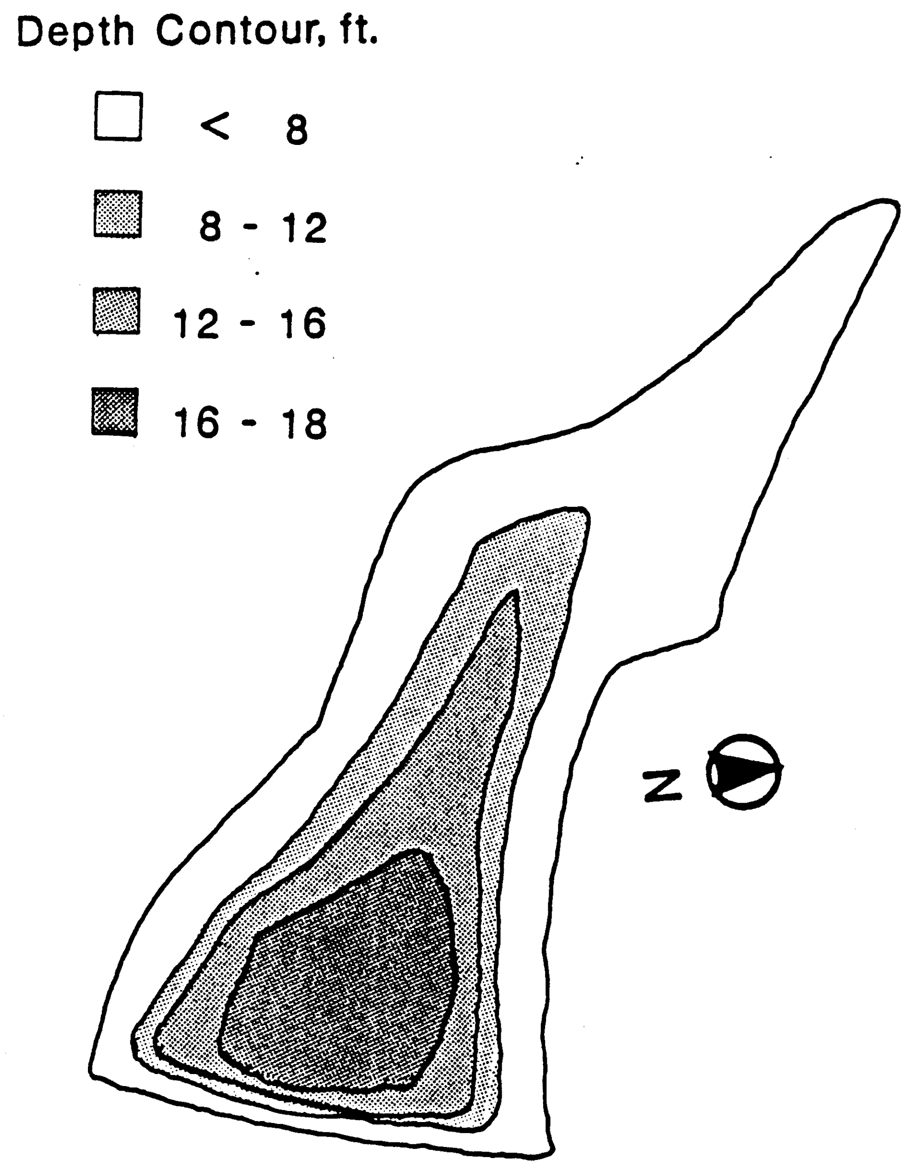


**Figure 6. Depth Contours of Lakeside Pond**

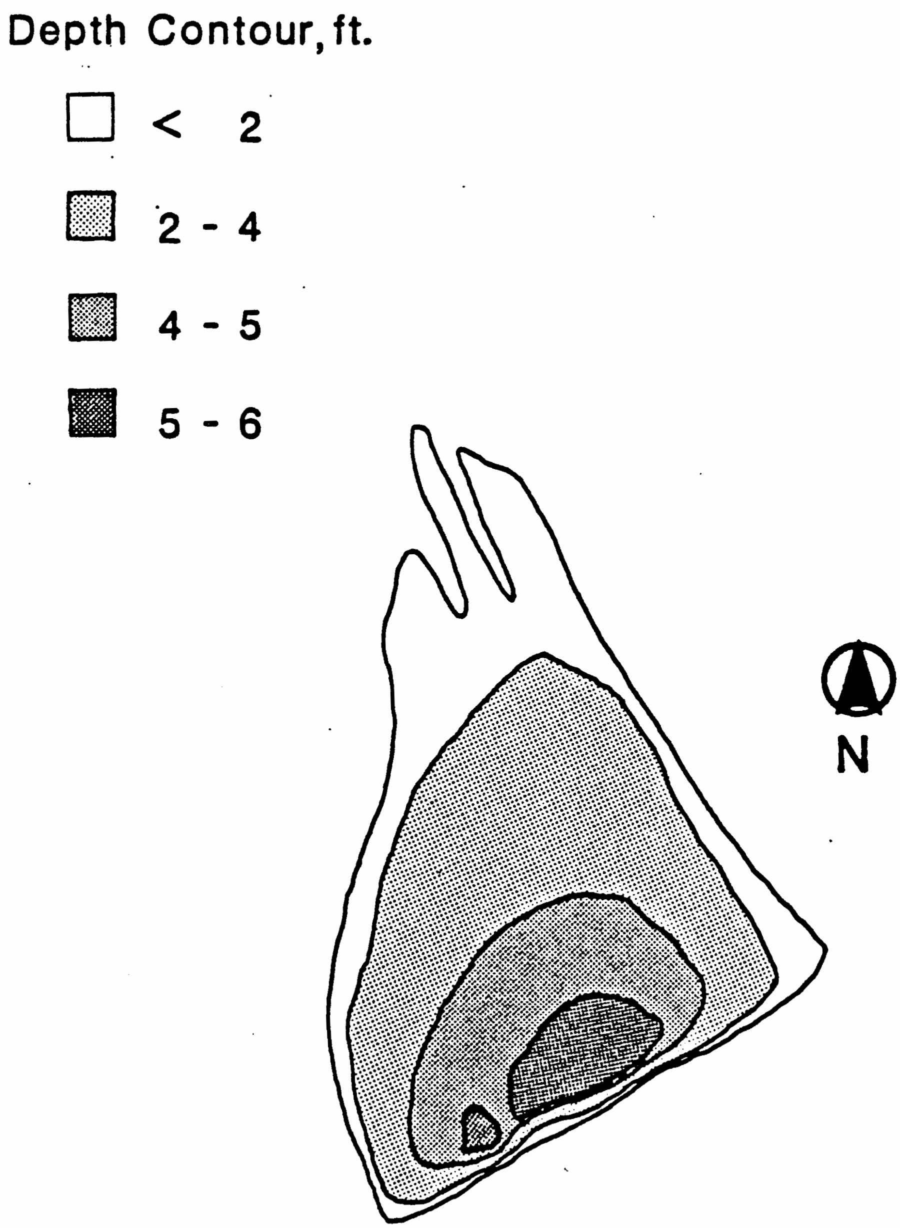


**Figure 7. Depth Contours of Waterford Pond**

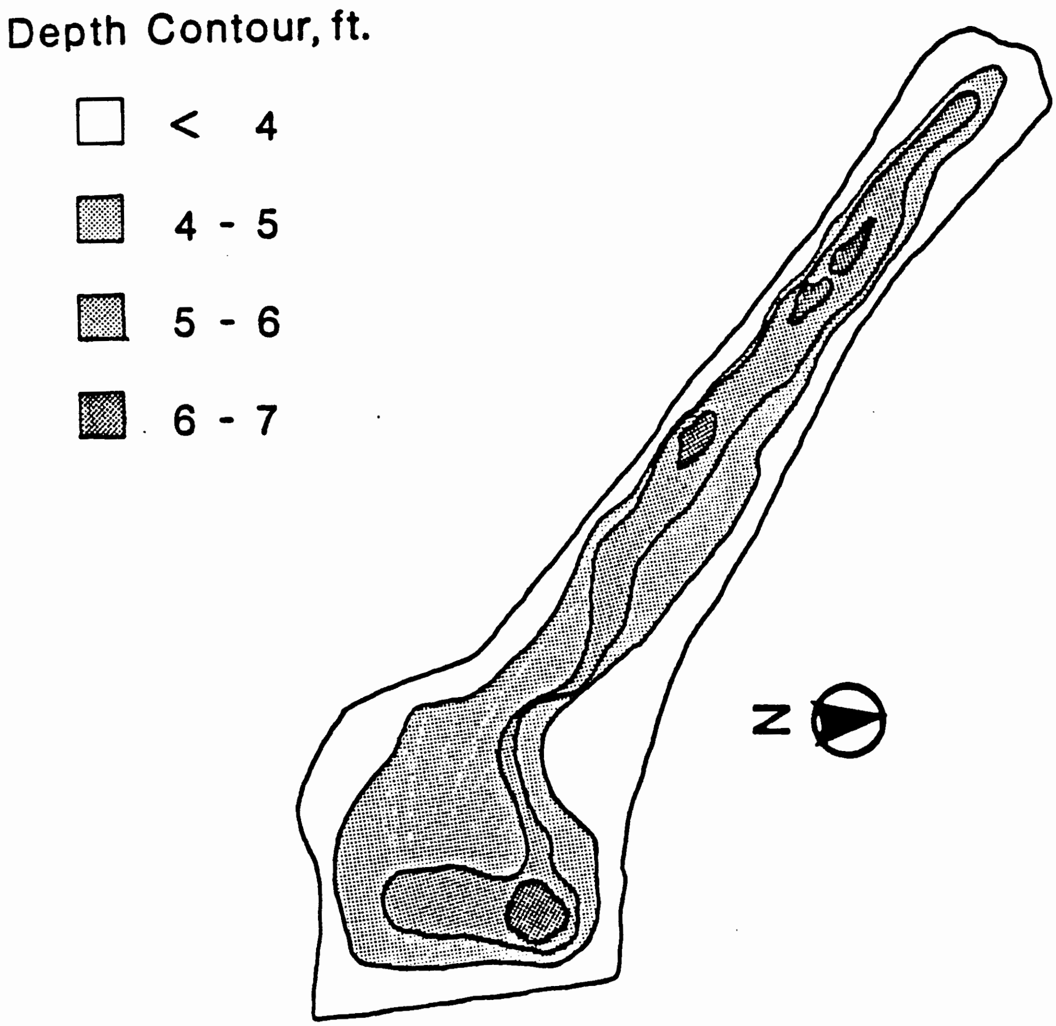


**Figure 8. Depth Contours of Runaway Bay Pond**

**Figure 9. General View of the Detention Pond (RB Pond)**

**Figure 10. Storm Pipe near Pond Outlet**

Figure11. Storm Pipe Draining Local Runoff From Area Surrounding the Detention Pond

Figure 12. Typical Outlet Weir of the Detention Ponds

5, 13 and 14). Rainfall increments and stream gage readings were recorded at 5-minute intervals on paper tapes. A tape reader transmits the records to the U.S. Geological Survey's Prime computer system for subsequent data processing.

Rating curves relating stream gage to flow were developed based in part on current-meter flow measurements at various stages. The resulting rating curve was combined with recorded stream stages to produce hydrographs at each gaging station. A generalized rating curve takes the form of :

$$q = K^*(H-a)^b \qquad (1)$$

where H is gage height in ft; and a, b and K are constants to be determined by fitting the rating curve to the respective flow and stage measurements. Table 2 presents the stage-discharge relationships for all five gaging stations.

For large storms, recorded stream stages might exceed the highest stage for which a current-meter measurement was made. In these cases, rating curves were extended using the conveyance-slope method (USGS, 1977; Wu, 1988).

**Estimation of Local Runoff**

Local runoff enters a detention pond as storm runoff originating from the surrounding impervious development. It is conveyed by storm pipes with diameters ranging from 6 inches to 2 feet (Figure 11). There are more than 15 storm pipes conveying local runoff into the LS and RB ponds. It was not possible to monitor all these local inflows. Therefore, two representative pipes from the Runaway Bay subarea were chosen for runoff estimation using the TR-20 hydrologic model (SCS, 1983). The contributing drainage for each pipe was determined. Based on estimated values of runoff curve number (CN) and time-of-concentration (Tc), the model computes the runoff hydrograph and volume from these areas. In order to obtain the total local runoff to a detention pond, the computed runoff volumes were extrapolated to the remaining pipes according to the proportion of drainage area. A simple calibration procedure, involving fine-tuning of the curve number, was employed to achieve a reasonable flow balance between the inflows (total local runoff plus upstream inflow at gaging station no. 3) and the downstream outflow from the RB pond. Table 3 summarizes the model parameters employed in the hydrologic simulation.

**Figure 13. Recording Stream Gaging Station**

Figure 14. Non-Recording Stream Gaging Station

### Table 2. Stage-Discharge Relationships

| Gaging Station No. | Staff Gage Height (H), ft | Equations |
|---|---|---|
| 1 | | Q= 0.299*(H+0.10)^4.700 |
| 2 | H<1.23 | Q=23.779*(H-0.24)^2.587 |
| | H>1.23 | Q=15.349*(H-0.02)^2.034 |
| 3 | H<2.24 | Q=11.194*(H-0.62)^2.237 |
| | H>2.24 | Q=14.952*(H-0.64)^1.745 |
| 4 | H<2.50 | Q=12.909*(H-0.92)^2.561 |
| | H>2.50 | Q=99.243*(H-2.10)^1.071 |
| 5 | H<1.73 | Q= 9.097*(H-0.40)^1.751 |
| | H>1.73 | Q=28.216*(H-1.20)^1.073 |

### Table 3. Runoff Curve Number and Time of Concentration for Runaway Bay Drainage Units

| | | Storm Pipe No. | |
|---|---|---|---|
| | | 1 | 2 |
| Contributing Drainage | | | |
| Area 1: | acres | 0.5 | 0.5 |
| | Tc, minutes | 1.5 | 1.5 |
| | CN | 98.0 | 98.0 |
| Area 2: | acres | 17.9 | 0.9 |
| | Tc, minutes | 6.0 | 3.0 |
| | CN | 88.0 | 88.0 |

## Water Quality Sampling

Stormwater samples were collected, using ISCO automatic samplers, from outflows of each detention pond (sampling sites 7, 8 and 11 shown in Figure 5). The sampling frequency varied from half an hour to two hours depending on whether samples were taken during or after a storm event. Normally, sampling of pond effluents lasted for about two days after the end of a storm event. In addition, runoff samples were collected manually at 10- to 20-minute intervals from the upstream inflow of LS pond (sampling site 6) and from the two selected storm pipes draining into RB pond (sampling sites 9 and 10).

Non-storm samples were collected once every two to three weeks at pond outlets to establish background water quality.

## Laboratory Procedures

Analyses of TSS, ammonia nitrogen ($NH_3$-N), total Kjeldahl nitrogen (TKN), orthophosphorus (OP), and total phosphorus (TP) were conducted in accordance with procedures outlined in Standard Methods (APHA, 1985). Metals were analyzed by the flame atomic absorption spectrophotometric method (AA). Dissolved metals were analyzed by filtering storm samples through a Nalgene syringe filter (cellulose acetate membrane) of 0.45-micron pore size. The filtrate was collected and acidified for subsequent analysis. Total metals were analyzed by digesting storm samples at 95 degrees centigrade overnight, using 2 ml of concentrated nitric acid per 50 ml of storm sample. The digested samples were then filtered for AA analysis.

The procedure for particle size analysis was similar to a settling column test with certain modifications. A number of graduated cylinders were filled with 500 ml storm samples of known initial TSS concentration. Initially, the content in each graduated cylinder was thoroughly mixed. At pre-determined time intervals, a 50-ml portion of the supernatant was withdrawn from the top of each cylinder for TSS analysis and, at the same time, the decrease in water level in the cylinder was recorded. The percentage of TSS remaining in the supernatant was the difference between the initial and subsequent TSS measurements. The distance that an average particle settled out of the 50-ml supernatant was taken as half of the recorded decrease in water level. The average settling velocity corresponding to the percentage of TSS remaining was obtained by dividing this distance by the time interval between sampling.

**Calculation of Pollutant Removal**

The amount of pollutants transported, M, was calculated by integrating the mass loading rate expressed as the product of flow, Q, and concentration, C, i.e.

$$M = \int Q*C \, dt \qquad (2)$$

The removal or trapping efficiency of pollutants was computed as the percent difference of the pollutant mass entering and leaving the detention pond.

$$E = \frac{M_{in} - M_{out}}{M_{in}} * 100\% \qquad (3)$$

Equation (2) provides a basis for computing the total mass of pollutants associated with the inflow of LS pond, and the outflows of RB, WF and LS ponds. Although outflows of RB and LS ponds were not directly monitored for flows; they could be indirectly obtained from the recorded flows at nearby stations. For instance, outflow of RB pond was read as the difference of the recorded flows at gaging stations 4 and 5.

The pollutant mass exported from local drainage areas was found by multiplying the areal runoff by an event mean concentration (EMC). EMC is defined as the constituent mass per unit runoff volume (mg/l) or a weighted average pollutant loading rate (lbs/acre/in runoff). The EMCs were derived from the runoff quality and flow information available at sampling sites 9 and 10 (storm pipes in the RB area).

Calculation of pollutant removal efficiency for the entire watershed was based on the principle of mass balance. An example of the computation is presented in Figure 15.

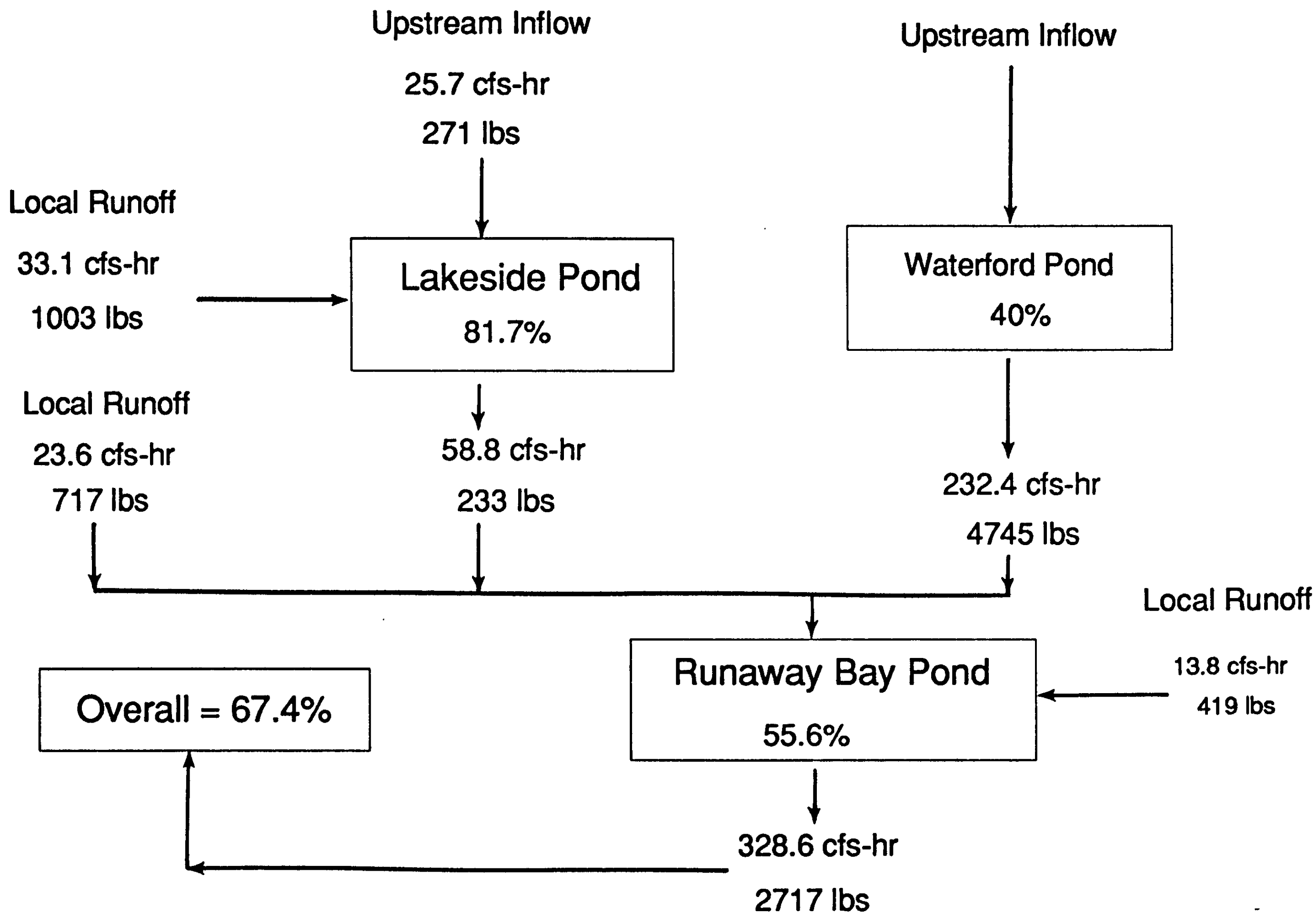


Figure 15. Example of TSS Removal Calculations (Storm 1)

## DATA PRESENTATION AND ANALYSIS

A total of eleven storm events was monitored. Table 4 summarizes rainfall records obtained from 11/18/86 to 12/01/87 and the annual precipitation statistics for North Carolina. The average monitored storm had a magnitude of 1.19 inches, a duration of 37 hours (1.6 days), and a time-since-last-storm of 233 hours (9.7 days).

Runoff samples were collected from the LS and RB pond areas for storms 1, 2, 3, 4, 5, 6, 7, and 10. This data base allows calculation of the pollutant removal efficiencies for LS pond (SA/DA=7.51), and RB pond (SA/DA=0.74). Runoff samples collected from the upstream inflow and downstream outflow of WF pond for storms 8, 9 and 11 were employed to calculate the removal efficiencies for WF pond (SA/DA=0.59). Based on the average performance of these three ponds, the overall performance of the ponding system (WF+RB+LS, SA/DA=2.27) can be derived.

### Hydrologic Performance

Tables 5, 6 and 7 present the hydrologic response of each detention pond. Since a large portion of the Lakeside drainage subarea is impervious, including streets and parking areas surrounding the pond, a higher runoff coefficient of 0.73 was observed. Computed runoff coefficients for Waterford and the entire watershed are 0.43 and 0.40, respectively. Runoff coefficients were computed by dividing the amount of outflow of a detention pond by the amount of runoff from the contributing drainage area.

The contribution of local runoff to each of the detention ponds is 60.3%, 0% and 8.4% for LS, WF and RB ponds, respectively. Local runoff into WF pond is negligible; its inflow is mainly from the upstream watershed runoff. On the other hand, RB pond receives the outflows from Lakeside (65 acres) and Waterford (302 acres) drainage subareas; its local runoff from the 70-acre adjacent impervious area accounts only for 8.4% of the combined inflow.

Poor attenuation of inflow peaks was observed for LS and RB ponds (Table 7). This is attributed to local inflows entering at or near the pond outlet, causing short circuiting of flow through the ponds (Figure 10). The effect of peak attenuation for WF pond was not quantified because gaging stations had not been installed to measure its upstream inflow. Nevertheless, outflow hydrographs of WF pond were back routed in order to estimate its inflow hydrographs for subsequent calculations of pollutant removal efficiency.

**Table 4. Rainfall Statistics of Monitored Storms**

| Storm No. | Date | Volume (in) | Duration (hrs) | Intensity (in/hr) | Time Since last Storm (hrs) |
|---|---|---|---|---|---|
| 1 | 01/01/87 | 1.39 | 17.8 | 0.078 | 184 |
| 2 | 02/26/87 | 3.60 | 66.7 | 0.054 | 89 |
| 3 | 03/25/87 | 1.73 | 146.8 | 0.012 | 137 |
| 4 | 04/15/87 | 1.48 | 80.0 | 0.019 | 383 |
| 5 | 06/04/87 | 0.64 | 2.8 | 0.226 | 9 |
| 6 | 09/05/87 | 1.06 | 23.6 | 0.045 | 16 |
| 7 | 10/27/87 | 0.52 | 8.3 | 0.062 | 647 |
| 8 | 11/10/87 | 1.08 | 6.8 | 0.160 | 329 |
| 9 | 12/10/87 | 0.56 | 3.7 | 0.153 | 273 |
| 10 | 02/02/88 | 0.80 | 42.8 | 0.019 | 149 |
| 11 | 02/19/88 | 0.26 | 11.4 | 0.023 | 352 |
| | Mean | 1.19 | 37.3 | 0.077 | 233 |
| | Cv | 0.77 | 1.2 | 0.919 | 0.8 |
| | | Rain Gage Records (11/18/86-12/01/87) | | | |
| | Mean | 0.50 | 6.6 | 0.222 | 112 |
| | Cv | 1.19 | 1.4 | 1.840 | 1.9 |
| | | N.C. Annual Statistics* | | | |
| | Mean | 0.36 | 5.9 | 0.066 | 77 |
| | Cv | 1.45 | 1.1 | 1.320 | 1.1 |

Cv = coefficient of variation
* U.S. EPA 1986.

**Table 5. Summary of Runoff Volume Calculations**

| Storm No. | LS Pond (1) | LS Pond (2) | LS Pond (3) | WF Pond (3) | RB Pond (1) | RB Pond (2) | RB Pond (3) |
|---|---|---|---|---|---|---|---|
| | Runoff Volume, cfs-hr | | | | | | |
| 1 | 25.7 | 33.1 | 82.4 | 232.4 | 314.8 | 13.8 | 328.6 |
| 2 | 98.6 | 137.5 | 334.2 | 882.6 | 1216.8 | 71.7 | 1288.5 |
| 3 | 23.0 | 44.5 | 99.4 | 191.5 | 290.9 | 32.5 | 323.7 |
| 4 | 17.1 | 26.6 | 62.8 | 180.8 | 243.6 | 26.3 | 269.9 |
| 5 | 5.0 | 11.5 | 8.3* | 22.6 | 30.9 | 3.6 | 34.3 |
| 6 | 10.4 | 12.6 | 9.6* | 37.6 | 47.1 | 8.2 | 55.3 |
| 7 | 4.1 | 5.1 | 3.8* | 15.3 | 19.2 | 3.3 | 22.5 |
| 8 | | | | 41.4 | | | |
| 9 | | | | 41.5 | | | |
| 10 | 22.9 | 26.9 | 69.0 | 135.9 | 204.9 | 17.0 | 222.0 |
| 11 | | | | 14.3 | | | |

(1) Upstream Inflow * Pond was not full at the beginning of storm.
(2) Local Runoff
(3) Downstream Outflow

**Table 6. Summary of Runoff Calculations**

| Storm No. | LS Pond | WF Pond | RB Pond |
|---|---|---|---|
| | Downstream Outflow, cfs-hr/in Rainfall | | |
| 1 | 42.3 | 167.2 | 236.4 |
| 2 | 65.6 | 245.2 | 357.9 |
| 3 | 39.0 | 110.7 | 187.1 |
| 4 | 29.5 | 122.2 | 182.4 |
| 5 | 0.0 | 35.3 | 53.6 |
| 6 | 0.0 | 35.5 | 52.2 |
| 7 | 0.0 | 29.4 | 43.3 |
| 8 | | 380.9 | |
| 9 | | 74.1 | |
| 10 | 62.5 | 169.6 | 277.5 |
| 11 | | 55.0 | |
| | Runoff Coefficient | | |
| Mean | 0.73 | 0.43 | 0.40* |
| | Percent Contribution of Local Runoff,% | | |
| Mean | 60.3 | 0.00 | 8.40 |

* Runoff coefficient for the entire watershed

**Table 7. Summary of Recorded Peak Discharges**

| Storm No. | LS Pond Inflow Qp | LS Pond Inflow Tp | LS Pond Outflow Qp | LS Pond Outflow Tp | WF Pond Outflow Qp | WF Pond Outflow Tp | RB Pond Inflow Qp | RB Pond Inflow Tp | RB Pond Outflow Qp | RB Pond Outflow Tp |
|---|---|---|---|---|---|---|---|---|---|---|
| 1 | 5.3 | 4.8 | 10.1 | 5.8 | 33.8 | 5.8 | 41.2 | 5.8 | 34.9 | 6.1 |
| 2 | 7.3 | 39.8 | 15.8 | 43.7 | 66.1 | 42.3 | 72.9 | 42.4 | 93.9 | 42.3 |
| 3 | 13.3 | 66.3 | 9.1 | 68.1 | 20.4 | 66.4 | 26.1 | 67.0 | 29.1 | 67.7 |
| 4 | 16.6 | 7.4 | 14.1 | 8.7 | 43.4 | 8.7 | 57.4 | 8.7 | 55.5 | 8.9 |
| 5 | 8.0 | 0.3 | 2.7 | 0.4 | 3.6 | 0.4 | 6.2 | 0.4 | 5.6 | 0.6 |
| 6 | 3.2 | 4.8 | 1.0 | 8.3 | 4.8 | 6.0 | 5.0 | 6.0 | 5.2 | 6.1 |
| 7 | 1.6 | 2.3 | 0.7 | 4.3 | 1.0 | 4.3 | 1.7 | 4.3 | 2.0 | 9.6 |
| 8 | | | | | 21.4 | 4.4 | | | | |
| 9 | | | | | 8.8 | 3.1 | | | | |
| 10 | 10.9 | 30.5 | 4.9 | 30.3 | 11.7 | 31.3 | 12.2 | 31.3 | 15.6 | 42.8 |
| 11 | | | | | 1.0 | 10.2 | | | | |

Qp = peak discharge in cfs measured at gaging station
Tp = time to peak, hrs

**Table 8. Event Mean Concentrations of Storm Runoff**

| Water Quality Parameters | Runaway Bay Local Runoff | NURP Values Residential | NURP Values Overall |
|---|---|---|---|
| TSS, mg/l | 135.00 | 228.00 | (1) |
| TKN, mg/l | 0.88 | 2.85 | 1.50 |
| $NH_3$-N, mg/l | 0.22 | | |
| TP, mg/l | 0.14 | 0.62 | 0.33 |
| OP, mg/l | 0.10 | | 0.12 |
| T-Fe, mg/l | 6.11 | | |
| S-Fe, mg/l | 0.24 | | |
| T-Cu, ug/l | (2) | 56.00 | 34.00 |
| T-Pb, ug/l | (2) | 293.00 | 144.00 |
| T-Zn, ug/l | 66.00 | 254.00 | 160.00 |
| S-Zn, ug/l | 20.00 | | |

(1) One order of magnitude or more greater than those from secondary treatment plant effluents
(2) Concentration below detection limit of flame atomic absorption spectrophotometric analysis
TP = total phosphorus OP = ortho phosphorus
T-Fe = total iron S-Fe = soluble iron

## Runoff Quality

The event mean concentration (EMC) of pollutant constituents in runoff was derived from data collected on the Runaway Bay local drainage areas (Table 8). The EMCs presented represent the average of data from three monitored storm events (storms 2, 3 and 4), and are representative of the runoff quality in the study area. In general, the EMCs obtained are lower than those reported in the NURP studies. TP and Zn are almost half and TKN is about 60% of the NURP values. The EMC of TSS is reported by NURP as one order of magnitude or more greater than those from secondary treatment plants. Taking a secondary effluent of 30 mg/l TSS, the NURP value for TSS would be 300 mg/l or greater. The average TSS obtained in this study is 135 mg/l. The EMCs for Cu and Pb are not available because their concentrations in runoff samples were below the sensitivity limit of the flame atomization analytical technique (30 ug/l for Cu and 100 ug/l for Pb). North Carolina water quality standards for aquatic life are 15 ug/l and 25 ug/l for Cu and Pb, respectively.

In addition, relationships between the total and soluble pollutant constituents in all runoff samples are included in Table 9.

## Particle Size Distribution

The performance of a detention pond is dictated by, among other factors, the particle size of incoming sediment. The particle size distribution may vary among storms, within a storm, and from one area to another. Consequently, ten sets of samples were collected from the Runaway Bay area during different storms and at various stages of storm events. The percentage of particles corresponding to an average settling velocity was analyzed. In order to compare the distributions, data for the 10 samples were averaged and are presented in Table 10 and Figure 16, along with NURP results. In general, the particle sizes encountered in this study area are much finer than the national average, e.g. the 50 percentile is about 1/3 finer than the national observation. The predominant soil in the Piedmont region of North Carolina consists mainly of clay and fine silt materials, which yield a finer particle size distribution in runoff. Since fine particles require a longer detention time to settle, this fact should be taken into account when detention ponds are to be sized using pollutant removal models.

**Table 9. Relationships between Soluble and Total Forms of Pollutant Parameters**

| | | |
|---|---|---|
| | Runaway Bay Local Runoff (Storm Pipe No. 2) | |
| OP = 0.805*TP - 0.013 | | NH3-N = 0.529*TKN - 0.228 |
| S-Zn = 0.100*Zn + 0.009 | | S-Fe = -0.006*Fe + 0.356 |
| | (Storm Pipe No. 2 ) | |
| OP = 0.328*TP + 0.039 | | NH3-N = 0.036*TKN + 0.046 |
| S-Zn = 0.257*Zn + 0.004 | | S-Fe = -0.007*Fe + 0.241 |
| | LS Inflow | |
| S-Zn = -0.18*Zn + 0.021 | | NH3-N = 0.176*TKN + 0.021 |
| | LS Outflow | |
| OP = 0.594*TP + 0.002 | | NH3-N = 0.447*TKN - 0.21 |
| S-Zn = 0.283*Zn + 0.001 | | S-Fe = 0.26*Fe + 0.042 |
| | WF Outflow | |
| OP = 0.554*TP - 0.002 | | NH3-N = 0.024*TKN + 0.03 |
| S-Zn = 0.036*Zn + 0.004 | | S-Fe = -0.002*Fe + 0.42 |
| | RB Outflow | |
| OP = 0.641*TP + 0.010 | | NH3-N = 0.109*TKN - 0.013 |
| S-Zn = 0.0004*Zn + 0.003 | | S-Fe = 0.043*Fe + 0.255 |

All units in mg/l except Zn in ug/l

**Table 10. Particle Settling Velocities For Runaway Bay Local Runoff**

| Size Category, microns | Percent of Particle in Each Category, % | Average Settling Velocity, ft/hr |
|---|---|---|
| 1 | 0 - 20 | 0.01 |
| 3 | 20 - 40 | 0.08 |
| 7 | 40 - 60 | 0.40 |
| 15 | 60 - 80 | 1.80 |
| 26 | 80 - 100 | 6.00 |

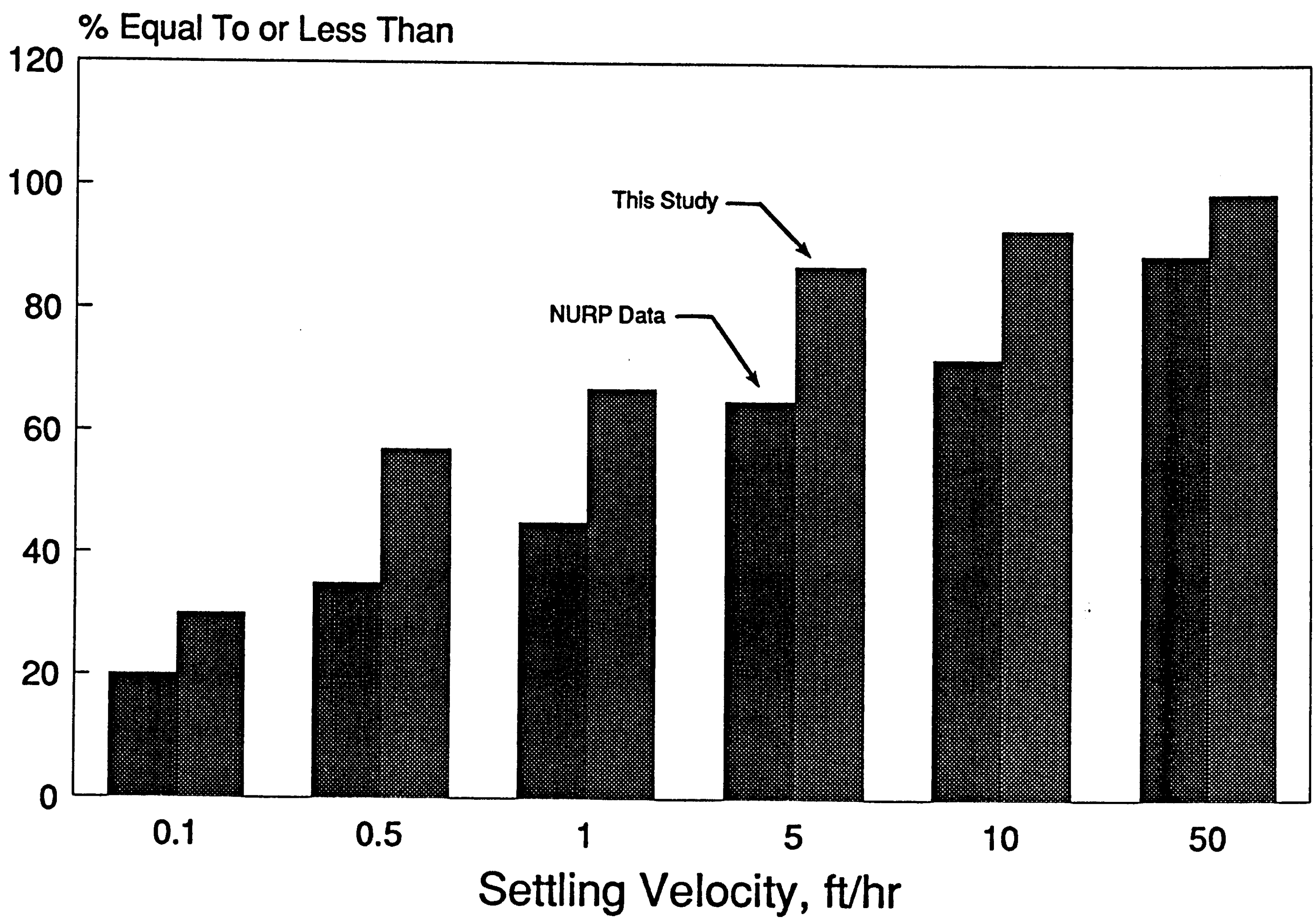


**Figure 16. Comparison of Settling Velocities in Runoff**

**Detention Pond Performance**

Tables 11 and 12 present the water quality response and removal efficiencies of detention ponds, respectively.

It can be seen from Table 11 that the average concentrations of WF pond outflow during storm events are generally higher than those of LS and RB ponds, particularly TSS. The average of peak concentrations from each storm event exceeds the background level. The average of mean event concentrations is not significantly higher than the background level, with the exception of TSS.

The removal efficiencies for RB pond averaged at 62% for TSS, 21 % for TKN, 36% for TP, 32% for Zn and 52% for Fe. The average TSS removal efficiency for WF pond is 41.4%. TSS is the only water quality parameter measured for WF pond.

LS pond, because of its large surface area ratio, performs well for the removal of TSS (93%), Zn (80%) and Fe (87%). A removal efficiency of 100% is reported for storms 5, 6 and 7 for all pollutant parameters. This is because the runoff produced by these small storms was totally retained in the LS pond, resulting in no overflow from the pond outlet. The removal efficiencies for TP (45%) and TKN (32%) are less than satisfactory probably due to the additional nutrient input from waterfowl droppings. It was estimated that 30 to 40 geese live in the LS pond area. The following computation estimates the relative contribution of nutrient loadings from urban geese in the Lakeside drainage subarea. Data pertinent to geese excrement are based on personal communication with Mr. John Dorney of the Division of Environmental Management, North Carolina Department of Natural Resources and Community Development.

<u>Assumptions</u>:

| | |
|---|---|
| Average annual rainfall, inches | 45.0 |
| Runoff Coefficient (Table 6) | 0.73 |
| TP loadings from runoff (Table 8) | |
| mg/l | 0.14 |
| lbs/ac-ft | 0.39 |
| TKN loadings from runoff (Table 8) | |
| mg/l | 0.88 |
| lbs/ac-ft | 2.39 |
| Number of geese | 35.0 |
| Daily excrement, lbs/100 geese | 2.70 |
| TP in excrement, % | 1.00 |
| TKN in excrement, % | 4.50 |

**Table 11. Water Quality Responses of Detention Ponds (LS Pond)**

| Storm No. | TSS mg/l | TP mg/l | TKN mg/l | Zn ug/l | Fe mg/l |
|---|---|---|---|---|---|
| | Storm Mean Concentration | | | | |
| (1) | | | | | |
| Inflow | 96 | 0.14 | 0.86 | 60 | |
| Outflow | 18 | 0.17 | 1.36 | 17 | |
| (2) | | | | | |
| Inflow | 98 | 0.14 | 0.86 | 60 | 5.5 |
| Outflow | 6 | 0.13 | 0.92 | 11 | 1.2 |
| (3) | | | | | |
| Inflow | 105 | 0.14 | 0.86 | 61 | 5.6 |
| Outflow | 6 | 0.03 | 0.94 | 18 | 1.3 |
| (4) | | | | | |
| Inflow | 101 | 0.14 | 0.86 | 61 | 5.5 |
| Outflow | 15 | 0.22 | 0.83 | 17 | 1.6 |
| (5) | | | | | |
| Inflow | 108 | 0.14 | 0.86 | 62 | 5.7 |
| Outflow | 0 | 0 | 0 | 0 | 0 |
| (6) | | | | | |
| Inflow | 95 | 0.14 | 0.86 | 60 | 5.4 |
| Outflow | 0 | 0 | 0 | 0 | 0 |
| (7) | | | | | |
| Inflow | 95 | 0.14 | 0.86 | 63 | 5.4 |
| Outflow | 0 | 0 | 0 | 0 | 0 |
| (10) | | | | | |
| Inflow | 95 | 0.14 | 0.86 | 59 | 5.4 |
| Outflow | 9 | 0.08 | 0.67 | 31 | 1.0 |
| Average | | | | | |
| Inflow | 99 | 0.14 | 0.86 | 61 | 5.5 |
| Outflow | 7 | 0.08 | 0.59 | 12 | 0.7 |
| | Storm Mean Peak Concentration | | | | |
| LS Pond Outflow | 17 | 0.25 | 1.5 | 70 | 2.2 |
| | Non-storm Mean Concentration | | | | |
| LS Pond Outflow | 6 | 0.15 | 1.2 | 45 | 0.9 |

Note: Inflow pollutant concentrations for each detention pond were obtained as flow-weighted average concentrations of its upstream inflow(s) and local runoff.

**Table 11. Water Quality Responses of Detention Pond (WF Pond) (Continued)**

| Storm No. | TSS mg/l | TP mg/l | TKN mg/l | Zn ug/l | Fe mg/l |
|---|---|---|---|---|---|
| | | Storm Mean Concentration | | | |
| (1) | | | | | |
| Outflow | 91 | 0.15 | 0.87 | 55 | |
| (2) | | | | | |
| Outflow | 40 | 0.11 | 0.58 | 55 | 4.9 |
| (3) | | | | | |
| Outflow | 16 | 0.04 | 0.80 | 5 | 1.4 |
| (4) | | | | | |
| Outflow | 41 | 0.29 | 0.66 | 16 | 4.5 |
| (5) | | | | | |
| Outflow | 66 | 0.11 | 1.28 | 32 | 3.2 |
| (6) | | | | | |
| Outflow | 51 | 0.14 | 0.80 | 32 | 3.5 |
| (7) | | | | | |
| Outflow | 13 | 0.01 | 0.43 | 29 | 0.9 |
| (8) | | | | | |
| Inflow | 341 | | | | |
| Outflow | 112 | | | | |
| (9) | | | | | |
| Inflow | 56 | | | | |
| Outflow | 34 | | | | |
| (10) | | | | | |
| Outflow | 5 | 0.06 | 0.38 | 47 | 2.5 |
| (11) | | | | | |
| Inflow | 14 | | | | |
| Outflow | 11 | | | | |
| Average | | | | | |
| Inflow | 137 | | | | |
| Outflow | 44 | 0.11 | 0.73 | 34 | 3.0 |
| | | Storm Mean Peak Concentration | | | |
| WF Pond Outflow | 108 | 0.25 | 1.3 | 120 | 8.0 |
| | | Non-storm Mean Concentration | | | |
| WF Pond Outflow | 9 | 0.15 | 1.4 | 36 | 1.5 |

**Table 11. Water Quality Responses of Detention Pond (RB Pond) (Continued)**

| Storm No. | TSS mg/l | TP mg/l | TKN mg/l | Zn ug/l | Fe mg/l |
|---|---|---|---|---|---|
| | | Storm Mean Concentration | | | |
| (1) | | | | | |
| Inflow | 83 | 0.15 | 0.96 | 49 | |
| Outflow | 37 | 0.06 | 0.94 | 27 | |
| (2) | | | | | |
| Inflow | 46 | 0.12 | 0.68 | 49 | 4.4 |
| Outflow | 50 | 0.13 | 0.49 | 63 | 4.3 |
| (3) | | | | | |
| Inflow | 37 | 0.06 | 0.85 | 20 | 2.3 |
| Outflow | 14 | 0.02 | 0.88 | 3 | 1.2 |
| (4) | | | | | |
| Inflow | 52 | 0.25 | 0.73 | 24 | 4.3 |
| Outflow | 14 | 0.21 | 0.46 | 8 | 1.9 |
| (5) | | | | | |
| Inflow | 90 | 0.12 | 1.15 | 44 | 4.2 |
| Outflow | 12 | 0.12 | 0.75 | 27 | 0.9 |
| (6) | | | | | |
| Inflow | 78 | 0.14 | 0.83 | 43 | 4.3 |
| Outflow | 18 | 0.09 | 0.57 | 49 | 1.4 |
| (7) | | | | | |
| Inflow | 51 | 0.05 | 0.57 | 42 | 2.6 |
| Outflow | 7 | 0.01 | 0.49 | 32 | 0.4 |
| (10) | | | | | |
| Inflow | 51 | 0.08 | 0.53 | 47 | 2.8 |
| Outflow | 22 | 0.05 | 0.42 | 28 | 2.0 |
| Average | | | | | |
| Inflow | 61 | 0.12 | 0.79 | 40 | 3.6 |
| Outflow | 22 | 0.08 | 0.63 | 30 | 1.7 |
| | | Storm Mean Peak Concentration | | | |
| RB Pond Outflow | 47 | 0.27 | 1.1 | 83 | 3.6 |
| | | Non-storm Mean Concentration | | | |
| RB Pond Outflow | 13 | 0.18 | 0.8 | 28 | 1.7 |

Note: Inflow pollutant concentrations for each detention pond were obtained as flow-weighted average concentrations of its upstream inflow(s) and local runoff.

**Table 12. Performance of LS, RB and WF Ponds**
(Field Data)

| Storm No. | TSS | TP | TKN | Zn | Fe |
|---|---|---|---|---|---|
| | | | Removal Efficiency, % | | |
| | | | LS Pond | | |
| 1 | 82 | -20 | -58 | 72 | - |
| 2 | 94 | 10 | -7 | 82 | 78 |
| 3 | 95 | 82 | -9 | 69 | 78 |
| 4 | 85 | -55 | 4 | 71 | 71 |
| 5 | 100 | 100 | 100 | 100 | 100 |
| 6 | 100 | 100 | 100 | 100 | 100 |
| 7 | 100 | 100 | 100 | 100 | 100 |
| 10 | 91 | 45 | 22 | 48 | 81 |
| Avg. | 93 | 45 | 32 | 80 | 87 |
| | | | RB Pond | | |
| 1 | 56 | 55 | 2 | 46 | - |
| 2 | -7 | -10 | 27 | -29 | 2 |
| 3 | 62 | 62 | -4 | 85 | 46 |
| 4 | 74 | 19 | 37 | 67 | 55 |
| 5 | 87 | -4 | 35 | 40 | 79 |
| 6 | 78 | 36 | 31 | -15 | 67 |
| 7 | 87 | 93 | 15 | 22 | 85 |
| 10 | 57 | 40 | 21 | 40 | 30 |
| Avg. | 62 | 36 | 21 | 32 | 52 |
| | | | WF Pond | | |
| 8 | 67 | | | | |
| 9 | 39 | | | | |
| 11 | 18 | | | | |
| Avg. | 41 | | | | |

Computation:

Annual TP contribution from geese :

(35 geese)(2.7 lbs/day-100 geese)(365 days)(1%)
= 3.45 lbs

Annual TKN contribution from geese :

(35 geese)(2.7 lbs/day-100 geese)(365 days)(4.5%)
= 15.52 lbs

Annual TP loadings from runoff :

(45 in)(0.73)(65 ac)(1/12 ft/in)(0.39 lbs/ac-ft)
= 69.40 lbs

Annual TKN loadings from runoff :

(45 in)(0.73)(65 ac)(1/12 ft/in)(2.39 lbs/ac-ft)
= 215.55 lbs

Therefore, geese droppings may contribute from 5% to 7% of the TP and TKN loadings to the LS pond, respectively. These figures suggest that nutrient contribution from urban geese may be less than 10% of the overall loading. The droppings are usually localized at a certain portion of the detention ponds. If this occurs near the outflow of a detention pond, a higher level of nutrients in the pond effluent could be expected.

## ASSESSMENT OF LONG TERM PERFORMANCE

In order to assess the long term performance of detention ponds, results of the field monitoring data from eleven storms were averaged and compared with the predicted performance as calculated by a computer model. The following is a brief description of the model and its applications.

### Modeling Methodology

The model for computing the long term removals of pollutants in detention ponds was developed by U.S. EPA (EPA, 1986). The analysis is based on the assumption that removal of sediment is due to the combined effect of dynamic settling during storm runoff and quiescent settling during the interval between successive storms. The variable nature of storm runoff is treated by specifying the rainfall and runoff in probabilistic terms, established by an appropriate analysis of a long term precipitation record.

(a) Removal under Dynamic Settling

Removal due to sedimentation in a dynamic, flow through system is represented by :

$$R = 1 - \left[ 1 + \frac{1}{n} * \frac{Vs}{Q/A} \right]^{-n} \qquad (4)$$

where R = fraction of solids removal
Vs = settling velocity of particles, ft/hr
Q/A = overflow rate, ft/hr
n = degree of short circuiting
(1 for very poor, 10 for ideal performance)

For a gamma distributed inflow entering a treatment device, the long term average removal is computed by :

$$RL = Z * \left[ \frac{r}{r - \ln(R/Z)} \right]^{r+1} \qquad (5)$$

where RL = long term average dynamic removal

R = removal at mean runoff rate from Equation (4)

Z = maximum fraction removal at very low flow rate

Cvq = coefficient of variation of runoff flow rates

r = $1/Cvq^2$

(b) Removal under Quiescent Conditions

The effective volume of a basin relative to the volume of runoff events is the principal factor determining removal effectiveness under quiescent conditions. Assuming a gamma distribution for storm volumes, the fraction of runoff that is not captured by a treatment device (Fv) is calculated as the expected value of the varying flow rates and duration. The solution has been obtained for a range of effective volume ratios (effective basin volume to mean runoff volume) and values for coefficient of variation in a range typically observed for rainfall/runoff. The effective volume ratio is expressed as a function of the solids removal rate (product of particle settling velocity and basin surface area), and the ideal basin ratio (actual basin volume divided by mean runoff volume). Thus, it is possible to calculate Fv by determining the solids removal rate and the actual basin ratio. The removal under quiescent condition (Rq) is given by:

$$Rq = 1 - Fv \qquad (6)$$

(c) Combined Removal

The removal efficiency under the combined effect of both dynamic and quiescent processes can be computed by applying the removal efficiency of either the dynamic or quiescent process to the pollutant fraction remaining after the operation of the other, i.e.

$$\text{Combined removal} = [\,1-(1-RL)*(1-Rq)] \qquad (7)$$

The following information is required to perform the computations.

1. Rainfall statistics including coefficient of variation for rainfall volume, intensity duration and interval.
2. Watershed characteristics including depth and surface area of the basin, and drainage area.
3. Runoff coefficient.
4. Short circuit parameter.
5. Particle size distribution.

## Model Applications

The EPA model was applied to the study area to compute the long term TSS removal. Rainfall records obtained from 11/18/86-12/01/87 (Table 4) were employed to derive input rainfall statistics. Watershed characteristics and runoff coefficients were obtained from Tables 1 and 6, respectively. A short-circuiting parameter of "3" was employed for all three ponds. This is based on the fact that the length-to-width ratios are higher than 2-to-1 for LS and RB ponds and is about 2-to-1 for the RB pond. Therefore, a relatively good geometry (n=3) for sedimentation can be assumed for these detention ponds. Input data of particle size distribution for LS and WF upstream inflows and for local runoff in the RB area are included in Table 10.

Initially, the model was run on the LS and WF ponds and the settling velocity distribution of each outflow was generated (Table 13). These distributions were combined, using a spreadsheet program, in proportion to their runoff volumes to derive a distribution for the upstream inflow of RB pond. Model calculation was then performed for RB pond using the combined distribution as an input along with other pertinent information.

Results of simulation for particle settling velocity and TSS removal efficiencies are presented in Tables 13 and 14, respectively. The computed TSS removal for LS, WF and RB ponds were 99%, 47% and 49%, respectively. The overall removal for the entire pond system (LS+WF+RB) was calculated to be 74%.

## Performance Assessment

The use of wet detention ponds or other stormwater treatment systems for controlling nonpoint pollution due to urban runoff is required by regulation in some cases for North Carolina coastal counties and encouraged in water supply watersheds.

In addition to ensuring proper hydraulic design, it is important for planners and design engineers to have data for initial sizing of detention ponds based on water quality considerations. The relationship between detention ponds performance and pond size needs to be refined. The field sampling program conducted in this study provides such data for performance evaluation. When comparing the field data for TSS removal with the predicted values, a good agreement is achieved as shown in Figure 17.

Pollutant parameters other than TSS could not be simulated by the model. Using Figure 1 as a guide, the removal of other pollutant constituents can be taken as fractions of TSS removal (TP removal is 66% of the reported TSS removal; TKN, Zn or Fe removal is

**Table 13. Computed Settling Velocities**
(EPA Model)

| Size Category | Percent Particle in Each Category | Average Settling Velocity in Outflow (ft/hr) | | |
|---|---|---|---|---|
| | | LS | WF | RB |
| 1 | 0 - 20 | 0.006 | 0.006 | 0.007 |
| 2 | 20 - 40 | 0.038 | 0.041 | 0.047 |
| 3 | 40 - 60 | 0.080 | 0.140 | 0.172 |
| 4 | 60 - 80 | 0.272 | 0.549 | 0.708 |
| 5 | 80 -100 | 0.960 | 3.185 | 3.305 |

Note: Simulation was based on rainfall records from 11/18/86-12/01/87 and data of the study site.

**Table 14. Computed TSS Removal Efficiency**
(EPA Model)

| Size Category | Percent Particle in Each Category | Removal Efficiency, % | | |
|---|---|---|---|---|
| | | LS | WF | RB |
| 1 | 0 - 20 | 99 | 9 | 11 |
| 2 | 20 - 40 | 99 | 38 | 48 |
| 3 | 40 - 60 | 99 | 44 | 51 |
| 4 | 60 - 80 | 100 | 58 | 58 |
| 5 | 80 -100 | 100 | 85 | 80 |
| | | ---- | ---- | ---- |
| | Overall | 99 | 47 | 49 |

Note: Simulation was based on rainfall records from 11/18/86-12/01/87 and data of the study site.

**Table 15. Relation Between Measured and Computed Removal Efficiency and Surface Area Ratio**

| SA/DA (%) | Pond System | Removal Efficiency, % TSS | TP | TKN | Zn | Fe |
|---|---|---|---|---|---|---|
| 0.59 | WF | 41 | 29 | 22 | 22 | 22 |
| | | (47) | (31) | (24) | (24) | (24) |
| 0.74 | RB | 62 | 36 | 21 | 32 | 52 |
| | | (49) | (32) | (25) | (25) | (25) |
| 2.27 | LS+WF+RB | 79 | 53 | 37 | 51 | 66 |
| | | (74) | (52) | (38) | (38) | (38) |
| 7.51 | LS | 93 | 45 | 32 | 80 | 87 |
| | | (99) | (65) | (50) | (50) | (50) |

Note: Measured removal obtained from Table 12.
Computed removal (Numbers in parenthesis) obtained from Table 14.

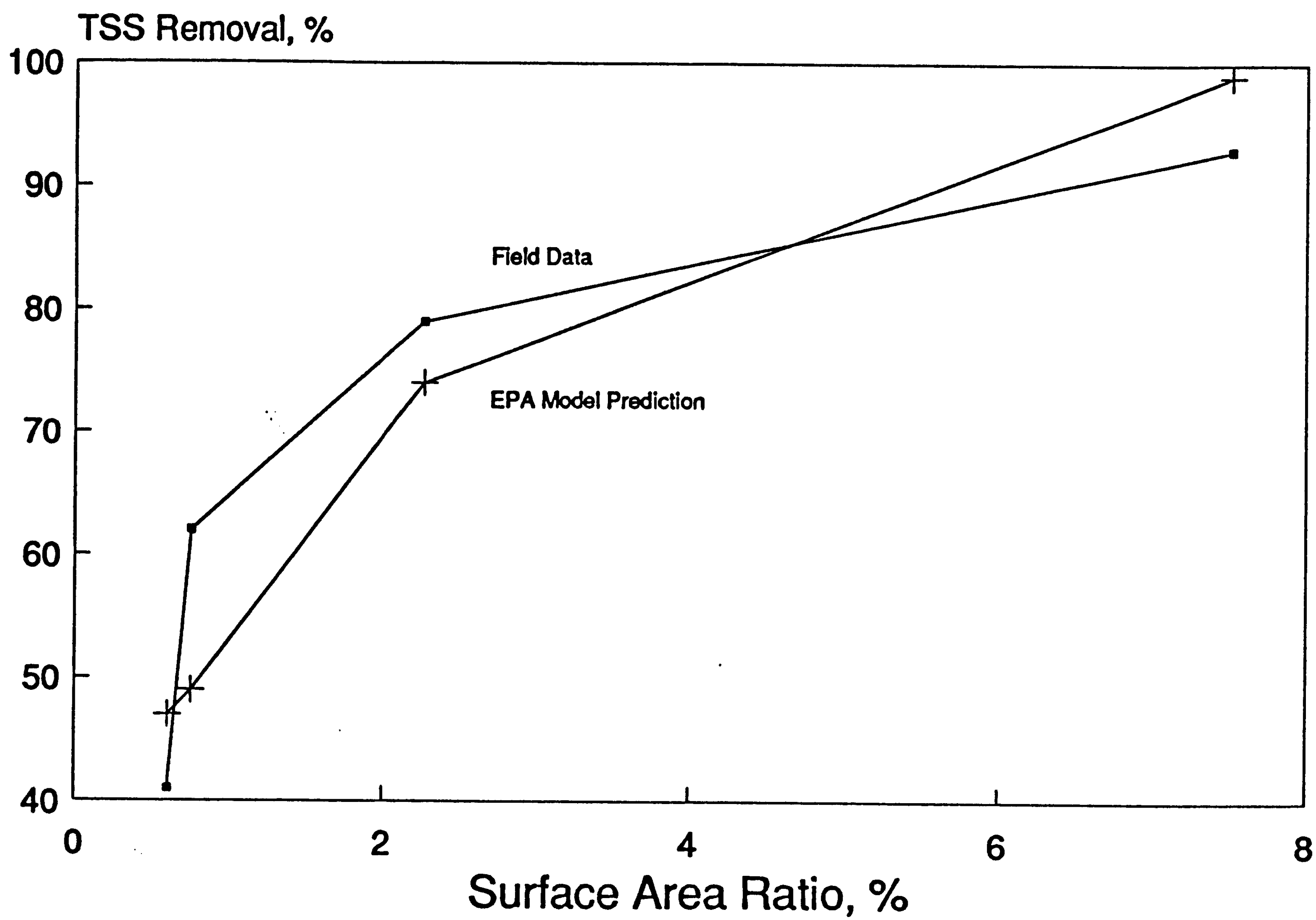


Figure 17. Comparison of TSS Removal

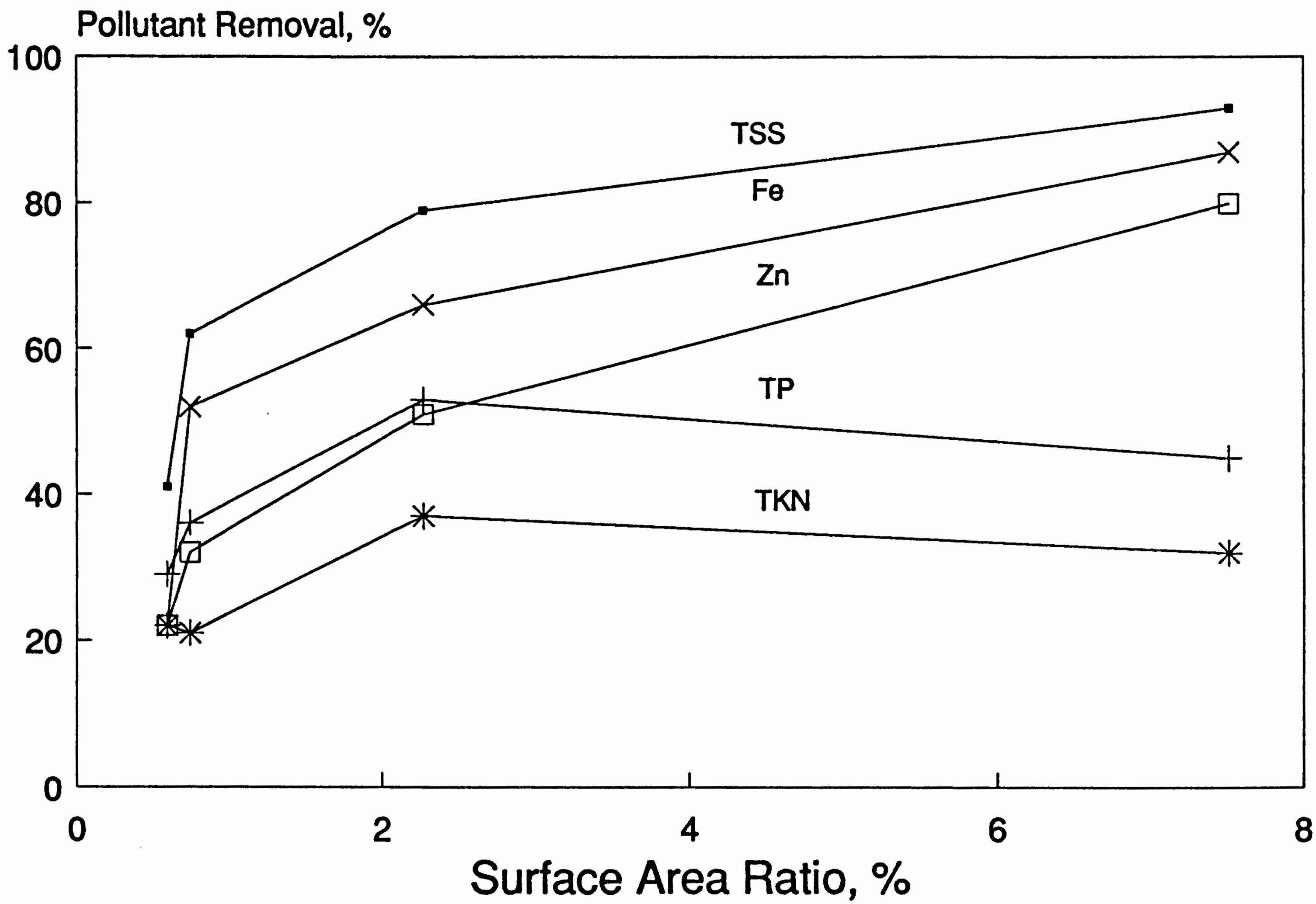


**Figure 18. Summary of Field Monitoring Data**

59% of the reported TSS removal). Consequently, the removal efficiencies for selected pollutant parameters as a function of surface area ratios can be established and presented in Table 15 and Figure 18.

The SA/DA ratios required to support 70% or better TSS removal would be 1% or greater. In order to provide 80% or better TSS removal, the SA/DA ratios would need to increase to 2% or greater (Figure 17 or 18). The North Carolina State Government criteria for 62% and 85% of TSS removal require SA/DA ratios ranged from 0.5-1.0% and 1.3-5.7%, respectively, depending on the basin depth and watershed characteristics. In comparison, the SA/DA ratios obtained from this research are 1% and 4.5% for achieving the targeted goals of 62% and 85% TSS removals, respectively. The results are reasonably consistent, implying that the North Carolina criteria are a reasonable planning tool for sizing detention ponds if local data are utilized in the design. These data include rainfall volume, intensity, duration, time-since-last-storm, runoff coefficient, and particle size distribution of sediment in storm runoff.

## LITERATURE CITED


APHA. 1985. Standard Methods for the Examination of Water and Wastewater. APHA, AWWA, WPCF.

ASCE. 1985. Stormwater Detention Outlet Control Structures. Task Committee on the Design of Outlet Control Structures, ASCE.

Camp, T.R. 1945. Sedimentation and the Design of Settling Tanks. Trans. ASCE, pp. 895-936.

Dally, L.K. 1983. Operation of Detention Facilities for Urban Stream Quality Enhancement. M.S. Thesis, University of Washington.

DEM. 1985. Toxic Substances in Surface Waters of the Falls and the Neuse Lake Watershed. Report no. 85-08. Division of Environmental Management, N.C. Department of Natural Resources and Community Development.

Griffin, D.M. Randall, C. and Grizzard, T.J. 1980. Efficient Design of Stormwater Holding Basins Used for Water Quality Protection. Water Research. Vol.14, pp.1549-1554.

Harrell, S.L. 1988. Multi-Model Micro-Computer Based Wet Detention Basin Design Methodology. In: Proceedings of Stormwater and Water Quality Model Users Group Meeting. U.S. EPA.

McCuen, R.H. 1980. Water Quality Trap Efficiency of Stormwater Management Basins. Water Research. Vol.1, pp.15-21.

Rausch, H.G. and Heinemann, H.G. 1975. Controlling Reservoir Trap Efficiency. Trans. ASCE, pp.1105-1113.

Striegl, R.G. 1987. Suspended Sediment and Metals Removal From Urban Runoff by a Small a Lake. Water Resources Bulletin. American Water Resources Association, Vol.23, No.4, pp.985-996.

Schueler, T.R., 1987. Controlling Urban Runoff: A Practical Manual for Planning and Designing Urban BMPs. U.S. Metropolitan Washington Council of Governments.

SCS. 1983. Computer Program for Project Formulation: Hydrology Technical Release 20. US Soil Conservation Service.

U.S. EPA, 1983. Results of Nationwide Urban Runoff Program-Executive Summary. US Environmental Protection Agency.

U.S. EPA, 1986. Methodology for Analysis of Detention Basins for Control of Urban Runoff Quality. EPA 440/5/87-001, US Environmental Protection Agency.

USGS. 1977. National Handbook of Recommended Methods for Water Data Acquisition. US Geological Survey.

Wu, J.S. and R.C. Ahlert, 1985. A Trajectory Model for Analyzing Sediment Trapping Efficiencies in Storm Water Detention Basins. In: Proceedings Conference on Stormwater and Water Quality Management Modeling. W. James (ed.) CHI Report R149. McMaster University, pp.257-263.

Wu, J.S. and R.C. Ahlert, 1986. Modeling Methodology for Dual-Function Stormwater Detention Basins. PB87-159711. National Technical Information Service.

Wu, C.J. 1988. Performance of Urban Wet Detentions. M.S. Thesis, University of North Carolina at Charlotte.