AN INTEGRATED METHODOLOGY FOR INSTREAM FLOW STRATEGIES

By

Miguel A. Medina, Jr.
Department of Civil and Environmental Engineering
School of Engineering
Duke University
Durham, North Carolina 27706

The work upon which this publication is based was supported in part by funds provided by the United States Department of the Interior, through the Water Resources Research Institute of The University of North Carolina as authorized by the Water Research and Development Act of 1978.

Project No. A-130-NC
Agreement No. 14-34-0001-2135
September 1983
DISCLAIMER STATEMENT

Contents of this publication do not necessarily reflect the views and policies of the U. S. Department of the Interior, nor does mention of trade names or commercial products constitute their endorsement or recommendation for use by the U. S. Government.
ACKNOWLEDGMENTS

This report is based on research supported partially by funds provided by the Office of Water Research and Technology, Department of the Interior, and the State of North Carolina through the Water Resources Research Institute of The University of North Carolina, Project No. A-130-NC, Agreement No. 14-34-0001-2135. The support and assistance provided by both the Director and the Associate Director, Drs. Dave Moreau and Jim Stewart, is gratefully acknowledged. Linda Kiger, Administrative Assistant, efficiently resolved budgetary matters.

The methodology and computer models presented in this report were applied to Salem Creek, Yadkin-Pee Dee River Basin, near Winston-Salem, North Carolina: the same site which was the subject of an earlier study, Hydrologic and Water Quality Modeling for Instream Flow Strategies, Report No. 183, December 1982. As acknowledged in that report, the professional staff of the U. S. Geological Survey Raleigh District Office and the Statesville Office furnished stage data, stage-discharge rating tables and other useful records and reports. Considerable assistance was also provided by staff members of the North Carolina Department of Natural Resources and Community Development. Water quality data were also obtained from records kept by the Archie Elledge Wastewater Treatment Plant.

Mr. Dennis Athayde, Chief of the Urban Nonpoint Sources Section, Implementation Branch, U. S. Environmental Protection Agency, Washington, D. C., provided the author with invaluable statistical analyses of both flows and pollutant concentrations, based on data collected nationwide under the National Urban Runoff Program (NURP). Dr. Gene Driscoll, E. D. Driscoll Associates, Oakland, New Jersey, deserves particular mention: he shared a number of microcomputer programs with the author, which were modified for use in this study. The author engaged in extensive discussion with Dr. Driscoll regarding the optimal mix of deterministically-based and statistically-based procedures. The author also wishes to acknowledge Dr. Dominic M. Di Toro, Manhattan College and Hydro Qual, Inc. (Mahwah, New Jersey) who developed several of the statistical methods adopted in this report. The professional contact between the author and these gentlemen over the past few years has resulted in a very positive technology transfer.

At Duke University, Mr. Fred Avent is credited with outstanding drafting support. Jo Ann Scott and Joan Lamorte typed the manuscript and assisted the author in the many tasks related to report preparation with great dedication. Mr. Neal Elliott, a graduate student, performed several programming tasks very well. Digital computations were performed through: (1) the Triangle Universities Computation Center, Research Triangle Park, North Carolina, through microwave links at the Duke University Computation Center, and (2) an IBM 5150 Personal Computer provided by the School of Engineering.
ABSTRACT

Instream flow assessments have traditionally resulted in the recommendation of a threshold value for the fishery resource: a minimum flow. More recently, an incremental methodology has been used to quantify the amount of potential habitat available for each life history state of a species as a function of streamflow. In a previous study, a framework was developed to address the impact of water quality fluctuations in determining instream flow strategies. Continuous hydrologic and water quality (transient) simulation models were applied to derive the frequency and duration of violations of established stream standards according to State stream use classifications.

In this study an integrated methodology is presented, based on a hierarchical package of computer models ranging from simple microcomputer programs to more complex mainframe simulation. The microcomputer programs are statistically-based, and assume that both flows and pollutant concentrations are lognormally-distributed for upstream sources, non-point sources and point sources. An extensive statistical analysis of nationwide data collected under the National Urban Runoff Program (NURP), as well as the study site for the selected time series (Salem Creek, Winston-Salem, N.C. -- November 1980 to August 1981), supports this assumption. Two programs are presented which compute the frequency distribution of pollutants in the stream based on lognormally-distributed point and nonpoint source flows and pollutant concentrations: (1) an approximate model by the method of moments and (2) a Gaussian quadrature, numerical solution model. The results are shown to be almost identical, but the approximate model executes much faster. The level of analysis proceeds to an intermediate stage: steady-state, continuous deterministic simulation. This level allows the representation of cause/effect (pollutant/water quality), coupled reactions. The predicted cumulative frequency curves for a selected pollutant (BOD) compare extremely well with the simple, statistical procedure. The methodology is extended to deterministic, continuous simulation of water quality transients. For a selected water quality indicator (DO), the steady-state model overpredicts the impact on receiving water quality up to about 5.5 mg/l (near the mid-point of the total spectrum of possible concentrations), then underpredicts slightly thereafter. Thus, the steady-state model is conservative in the range of concentrations up to slightly beyond the stream standard. The more sophisticated model also provides the frequency distribution of consecutive hours of violation of a stream standard, and allows viewing both cause/effect relationships in time and space through 3-dimensional computer graphics.
# TABLE OF CONTENTS

| ACKNOWLEDGMENTS | ................................................................. | iii |
| ABSTRACT | .......................................................... | v |
| LIST OF FIGURES | .................................................. | ix |
| LIST OF TABLES | .................................................. | xvii |
| SUMMARY, CONCLUSIONS AND RECOMMENDATIONS | .............................................. | xx |

## CHAPTER I. INTRODUCTION AND OVERVIEW OF INTEGRATED METHODOLOGY ......................................................... 1

1.1 Introduction ...............................................................
1.2 Overview of Integrated Methodology .............................

## II. FREQUENCY ANALYSIS IN HYDROLOGIC AND WATER QUALITY MODELING ..................................................... 10

2.1 Historical Review ..................................................
2.2 Hydrologic Frequency Studies ................................
2.3 Water Quality Frequency Curves ................................
2.4 Deterministic Simulation Versus Derived Distribution Approaches ........................................

## III. METHODOLOGY ................................................................ 23

3.1 Rainfall Time Series Analysis ......................................
3.2 Hydrologic Modeling ...................................................
3.3 Deterministic Water Quality Modeling ..........................
3.4 Key Deterministic Water Quality Model Parameters ....
3.5 Statistical Water Quality Models .................................

## IV. DESCRIPTION OF STUDY AREA AND HYDROLOGIC TIME SERIES ............................................................. 51

4.1 Yadkin-Pee Dee River Basin ......................................
4.2 Climate .................................................................
4.3 Water Quality of Yadkin-Pee Dee River, North Carolina ..........................
4.4 Salem Creek and Muddy Creek .................................
## LIST OF FIGURES

<table>
<thead>
<tr>
<th>Figure</th>
<th>Description</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>I-1</td>
<td>Weighted Usable Area Versus Discharge for Smallmouth Bass</td>
<td>3</td>
</tr>
<tr>
<td>I-2</td>
<td>Monthly Weighted Usable Area Values for Adult Smallmouth Bass Under Median and Drought Year Flow Conditions</td>
<td>4</td>
</tr>
<tr>
<td>II-1</td>
<td>The Probability Density Function and its Cumulative Distribution Function</td>
<td>14</td>
</tr>
<tr>
<td>II-2</td>
<td>Flow-Duration Curve for Hypothetical Watershed</td>
<td>14</td>
</tr>
<tr>
<td>II-3</td>
<td>Cumulative Water Quality Frequency Curves for Hypothetical Watershed</td>
<td>16</td>
</tr>
<tr>
<td>II-4</td>
<td>Frequency Distribution of Consecutive Time Periods of Violation of the Selected Stream Standard</td>
<td>16</td>
</tr>
<tr>
<td>II-5</td>
<td>Normal Probability Plots of Logarithms of Upstream Flows and Stormwater Surface Runoff</td>
<td>19</td>
</tr>
<tr>
<td>II-6</td>
<td>Normal Probability Plots of Logarithms of Stormwater Organic Pollutant Concentrations</td>
<td>19</td>
</tr>
<tr>
<td>II-7</td>
<td>Normal Probability Plot of Logarithms of Stormwater Fecal Coliform Concentrations</td>
<td>20</td>
</tr>
<tr>
<td>II-8</td>
<td>Normal Probability of Logarithms of Stormwater Total suspended Solids Concentrations</td>
<td>20</td>
</tr>
<tr>
<td>II-9</td>
<td>Normal Probability Plots of Logarithms of Stormwater Nitrogen Concentrations</td>
<td>21</td>
</tr>
<tr>
<td>II-10</td>
<td>Normal Probability Plots of Logarithms of Stormwater Phosphorus Concentrations</td>
<td>21</td>
</tr>
<tr>
<td>II-11</td>
<td>Normal Probability Plot of Logarithms of Stormwater Lead Concentrations</td>
<td>22</td>
</tr>
<tr>
<td>II-12</td>
<td>Normal Probability Plots of Logarithms of Stormwater Toxics Concentrations, Copper and Zinc</td>
<td>22</td>
</tr>
</tbody>
</table>
III-1. A Generalized Component of the Physical System .............................................. 34

III-2. Deoxygenation Rate As a Function of Stream Depth (modified after Hydroscience, Inc., 1971). Equation shown is a specific function which corresponds to $\gamma_1 = 0.99$ and $\gamma_2 = 0.28$ .......... 43

IV-1. Yadkin-Pee Dee River Basin, North Carolina, South Carolina and Virginia ............................................ 52

IV-2. The Main Branch of the Yadkin-Pee Dee River in North Carolina .................................................. 52


IV-4. Land Resources Information Service Land Use Analysis for the Salem Creek Drainage Area ...... 65

IV-5. Population Projection for Winston-Salem/Forsyth County, 1975-1985 ............................................. 69


IV-7. Rainfall and Evaporation Stations Nearest Winston-Salem, North Carolina .............................. 71

IV-8. Streamflow and Water Quality Monitoring Stations, Salem Creek and Muddy Creek Drainage Basin .................................................. 71

IV-9. Salem Dam and Lake .......................................................... 72

IV-10. Salem Creek Flowing Under U.S. Highway 52 Bridge and Railroad Bridge ............................. 73

IV-11. NURP Rain Gages in the Central Business District and Main Street Bridge .............................. 74

IV-12. USGS Streamflow Station No. 02115856, Salem Creek .. 75
| IV-13.  | Salem Creek Near Archie Elledge Wastewater Treatment Plant, City of Winston-Salem | 76 |
| IV-14.  | USGS Streamflow Station No. 02115860, Muddy Creek | 77 |
| IV-15.  | USGS Fluorescent Dye Study Reach, Salem Creek to Yadkin River | 78 |
| IV-16.  | Yadkin River at Robinhood Road and the Shalloford Road Bridge, Winston-Salem | 80 |
| IV-17.  | Yadkin River near Winston-Salem, North Carolina | 81 |
| IV-18.  | Average Annual Frequency Histogram of Hourly Rainfall, NOAA Station No. 313630 | 82 |
| IV-19.  | Frequency Histogram of Hourly Rainfall for 1974, NOAA Station No. 313630 | 82 |
| IV-20.  | Correlogram of Hourly Rainfall for 1974, NOAA Station No. 313630 | 83 |
| IV-21.  | Storm Volumes Versus Month of Year, 1948-1975, NOAA Station No. 313630 | 85 |
| IV-22.  | Storm Duration Versus Month of Year, 1948-1975, NOAA Station No. 313630 | 85 |
| IV-23.  | Storm Intensities Versus Month of Year, 1948-1975, NOAA Station No. 313630 | 86 |
| IV-24.  | Time Since Previous Storm, 1948-1975, NOAA Station No. 313630 | 86 |
| IV-25.  | Recurrence Interval Versus Storm Volume, 1948-1975, NOAA Station No. 313630 | 87 |
| IV-26.  | Storm Duration Versus Recurrence Interval, 1948-1975, NOAA Station No. 313630 | 87 |
| IV-27.  | Storm Average Intensity Versus Recurrence Interval, 1948-1975, NOAA Station No. 313630 | 88 |
| IV-28.  | Recurrence Interval Versus Time Since Previous Storm, 1948-1975, NOAA Station No. 313630 | 88 |
IV-29. Average Storm Depth In Each Year of Record, NOAA Station No. 313630 .................. 89
IV-30. Average Duration In Each Year of Record, NOAA Station No. 313630 .................. 89
IV-31. Average Rainfall Intensity In Each Year of Record, NOAA Station No. 313630 ............ 90
IV-32. Average Time Since Previous Storm For Each Year of Record, NOAA Station No. 313630 ....... 90
IV-33. Cumulative Frequency Curve for Rainfall Depth, 1948-1975, NOAA Station No. 313630 ........ 91
IV-34. Cumulative Frequency Curve for Storm Duration, 1948-1975, NOAA Station No. 313630 ........ 91
IV-35. Cumulative Frequency Curve for Average Rainfall Intensity, 1948-1975, NOAA Station No. 313630 ........ 92
IV-36. Cumulative Frequency Curve for Time Since Previous Storm, 1948-1975, NOAA Station No. 313630 .... 92
IV-37. A Workman Removes Dead Fish From the City of Salisbury's Water Supply Reservoir, Near Ellis Crossroads, North Carolina .................. 95
IV-38. A Closeup of Dead Fish Floating In the City of Salisbury Reservoir .................. 96
<table>
<thead>
<tr>
<th>Page</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>99</td>
<td>Normal Probability Plots of Logarithms of Stormwater Nitrogen Concentrations, Winston-Salem NURP Sites</td>
</tr>
<tr>
<td>100</td>
<td>Normal Probability Plots of Logarithms of Stormwater Zinc and Lead Concentrations, Winston-Salem NURP Sites</td>
</tr>
<tr>
<td>102</td>
<td>Streamflow and Water Quality Monitoring Stations, Salem Creek and Muddy Creek, North Carolina</td>
</tr>
<tr>
<td>103</td>
<td>Archie Elledge WWTP Upstream Monitoring Station, Winston-Salem, N.C.</td>
</tr>
<tr>
<td>103</td>
<td>Archie Elledge WWTP Downstream Monitoring Station, Winston-Salem, N.C.</td>
</tr>
<tr>
<td>105</td>
<td>Linear Regression of Log-Normally Distributed Upstream Flows, Salem Creek, Near Winston-Salem, N.C.</td>
</tr>
<tr>
<td>105</td>
<td>Linear Regression of Log-Normally Distributed Upstream BOD Concentrations, Salem Creek, near Winston-Salem, N.C.</td>
</tr>
<tr>
<td>106</td>
<td>Linear Regression of Log-Normally Distributed Archie Elledge WWTP Effluent Flows, Salem Creek</td>
</tr>
<tr>
<td>106</td>
<td>Linear Regression of Log-Normally Distributed Archie Elledge WWTP Effluent BOD Concentrations, Salem Creek</td>
</tr>
<tr>
<td>107</td>
<td>Linear Regression of Log-Normally Distributed Storm and Point Flows, Salem Creek, near Winston-Salem, N.C.</td>
</tr>
<tr>
<td>107</td>
<td>Linear Regression of Log-Normally Distributed Storm and Point BOD Concentrations, Salem Creek, near Winston-Salem, N.C.</td>
</tr>
<tr>
<td>114</td>
<td>SAS UNIVARIATE Normal Probability Plot On Log-Upstream Flows, Salem Creek</td>
</tr>
<tr>
<td>114</td>
<td>SAS UNIVARIATE Normal Probability Plot On Log-Upstream BOD Concentrations, Salem Creek</td>
</tr>
</tbody>
</table>
V-12. SAS UNIVARIATE Normal Probability Plot On Log-Storm and Point Flows, Salem Creek .................. 115

V-13. SAS UNIVARIATE Normal Probability Plot On Log-Storm and Point Source BOD Concentrations, Salem Creek .................. 115

V-14. SAS UNIVARIATE Frequency Distribution Bar Chart On Log-Storm and Point Source BOD Concentrations, Salem Creek .................. 116

V-15. SAS UNIVARIATE Normal Probability Plot On Log-Point Source Flows, Salem Creek .................. 116

V-16. SAS UNIVARIATE Normal Probability Plot On Log-Point Source BOD Concentrations, Salem Creek .... 117

V-17. Muddy Creek River Basin Discretization for Hydraulic Stimulation ........................................ 117

V-18. Discretization of Salem Creek (Kerners Mill Creek) Above Salem Lake and Dam ........ 118

V-19. Discretization of Salem Creek Into Overland Flow and Stream Flow Segments .................. 118

V-20. Salem Creek Calibration, USGS Distributed Routing Rainfall-Runoff Model .................. 122

V-21. Salem Creek Verification, USGS Distributed Routing Rainfall-Runoff Model .................. 122

V-22. Predicted Stream BOD Concentration Frequency Distribution, Moments Approximation and Gaussian Quadrature .................. 123


V-25. Comparison of Deterministic Steady-State Versus Transient Simulation, Salem Creek .................. 129

V-27. BOD Concentration in Time and Space, July 16 - August 3, 1981, Salem Creek ............................................................... 131

V-28. DO Concentration in Time and Space, July 16 - August 3, 1981, Salem Creek ............................................................... 131

V-29. BOD Concentration in Time and Space, November 15 - November 29, 1980, Salem Creek ...................................................... 132

V-30. DO Concentration in Time and Space, November 15 - November 29, 1980, Salem Creek ...................................................... 132

V-31. Frequency Distribution of Duration of Violations of the Stream DO Standard, Salem Creek ....................................................... 133

V-32. Effects of Chronic and Acute Exposure of Non-salmonid Fish to Dissolved Oxygen Levels (modified after Driscoll, 1981) ..................... 133
# LIST OF TABLES

<table>
<thead>
<tr>
<th>Table</th>
<th>Title</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>I-1</td>
<td>Hierarchical Approach For Instream Water Quality Analysis</td>
<td>8</td>
</tr>
<tr>
<td>IV-1</td>
<td>Major Municipal and Industrial Wastewater Discharges To the Yadkin-Pee Dee River (modified from North Carolina Department of Natural and Economic Resources, 1976)</td>
<td>58</td>
</tr>
<tr>
<td>IV-2</td>
<td>Classifications of Waters in North Carolina (Division of Environmental Management, 1979)</td>
<td>60</td>
</tr>
<tr>
<td>IV-3</td>
<td>State of North Carolina Freshwater Stream Standards (Division of Environmental Management, 1979; U.S. Water Resources Council, 1979)</td>
<td>52</td>
</tr>
<tr>
<td>IV-4</td>
<td>Selected Yadkin River and Tributary Stream Use Classifications, North Carolina (STORET, 1981)</td>
<td>63</td>
</tr>
<tr>
<td>IV-5</td>
<td>Land Resources Information Service (LRIS) Land Use Analysis for Salem Creek Drainage Basin (LRIS, 1981)</td>
<td>66</td>
</tr>
<tr>
<td>IV-6</td>
<td>Land Use Projections for Upper Yadkin, % (modified after U.S. Water Resources Council, 1979)</td>
<td>68</td>
</tr>
<tr>
<td>IV-7</td>
<td>Population Projections for Forsyth County</td>
<td>63</td>
</tr>
<tr>
<td>V-1</td>
<td>SAS UNIVARIATE Statistics On Log-Upstream Flows, Salem Creek, Winston-Salem, North Carolina</td>
<td>108</td>
</tr>
<tr>
<td>V-2</td>
<td>SAS UNIVARIATE Statistics On Log-Upstream BOD Concentrations, Salem Creek, Winston-Salem, North Carolina</td>
<td>109</td>
</tr>
<tr>
<td>V-3</td>
<td>SAS UNIVARIATE Statistics On Log-Storm and Point Flows, Salem Creek, Winston-Salem, North Carolina</td>
<td>110</td>
</tr>
<tr>
<td>V-4</td>
<td>SAS UNIVARIATE Statistics on Log-Storm and Point BOD Concentrations Salem Creek, Winston-Salem, North Carolina</td>
<td>111</td>
</tr>
<tr>
<td>V-5</td>
<td>SAS UNIVARIATE Statistics On Log-Point Source Flows, Salem Creek, Winston-Salem, North Carolina</td>
<td>112</td>
</tr>
</tbody>
</table>
V-6. SAS UNIVARIATE
Statistics on Log-Point Source BOD Concentrations,
Salem Creek, Winston-Salem, North Carolina ................. 113

V-7. Overland Flow Segment Areas for
Salem Creek Above and Below Salem Dam
and Lake ...................................................... 120

V-9. Predicted Output from Moments Approximation
Model, COD, Salem Creek ..................................... 125

V-10. Predicted Output from Moments Approximation Model,
Fecal Coliform, Salem Creek .................................. 126

V-11. Lane Use Pollutant Accumulation Rates ..................... 127

V-12. Average Values of Key STO/TRT and Level III -
RECEIVING Parameters and Hydraulic Variables .............. 128
An integrated methodology for instream flow strategies is presented, based on a hierarchical package of computer models ranging from simple microcomputer programs to more complex mainframe simulation. The microcomputer programs are statistically-based, and assume that both flows and pollutant concentrations are lognormally-distributed for upstream sources, non-point sources and point sources. An extensive statistical analysis of nationwide data collected under the National Urban Runoff Program (NURP), as well as at the study site for the selected time series (Salem Creek, Winston-Salem, N.C.—November 1980 to August 1981), supports this assumption. The level of analysis progresses to an intermediate level: steady-state, continuous deterministic simulation. This level allows the representation of cause/effect (pollutant/water quality), coupled reactions. The predicted cumulative frequency curves for a selected pollutant (BOD) compare extremely well with the simple, statistical procedure. The methodology is extended to deterministic, continuous simulation of water quality transients. For a selected water quality indicator (DO), the steady-state model overpredicts the impact on receiving water quality up to about 5.5 mg/l (near the mid-point of the total spectrum of possible concentrations), then underpredicts slightly thereafter. Thus, the steady-state model is conservative in the range of concentrations up to slightly beyond the stream standard. The more sophisticated model also provides the frequency distribution of consecutive hours of violation of a stream standard, and allows viewing both cause/effect relationships in time and space through 3-dimensional computer graphics.

Rainfall and, therefore, surface runoff (streamflow) are both inherently random events. It is appropriate to analyze water quality effects for instream flow strategies within a probabilistic setting. It does not follow, however, that the mathematical models themselves should be exclusively statistical black boxes. Indeed, the response of receiving waters to pollutant mass discharges may and has been described by deterministic mass balance models. Continuous, long term deterministic simulation allows representation of water quality effects in terms of the probability of occurrence of events of various magnitudes. Stochastic inputs to such a deterministic model will result in random output. Probabilistic analysis attempts to calculate the probability distribution of receiving water quality concentrations given the probability distribution of model inputs: for example, hydrology, pollutant loadings, stream temperature, etc. A framework is presented in which both statistical and deterministic models can be integrated in the analysis to achieve: long-term characterization of the rainfall-runoff process and derivation of both hydrologic and water quality frequency distributions. If the level at which a particular pollutant concentration impairs the use of the receiving stream is known, then the concentration probability distribution specifies the frequency (and duration in the case of simulation of transients) with which that use (fisheries, waste allocation) will be impaired.
If the assumptions upon which the statistical models are based are met, for a particular study site and hydrologic time series, the calculations can be easily performed on a microcomputer and the data requirements are relatively simple. These models are very useful for preliminary water resource assessment. They can be followed by a more refined approach if it is justified. Continuous deterministic simulation (both steady-state and transient) accounts for the actual sequencing of storm event loads. Much more complex statistical procedures (often impractical) would be required to approximate this phenomena. Deterministic models also produce a more complete time history of system response to excitation, representing not only individual storm properties but also cumulative effects of closed-spaced events. These models can be relatively expensive to execute, are data intensive and require large memory devices for data manipulation and storage. Advances in microcomputer technology are making it possible to load these large programs into memory but execution times are still extremely long.

Synoptic rainfall data analysis of a long-term record (25-year) of hourly values allows statistical characterization of important storm event variables: average intensity, volume, duration and time since the previous event. Two microcomputer programs are provided to analyze the flows and pollutant concentrations for lognormality. One program computes the parameters of a two-parameter lognormal distribution by the method of moments and the method of maximum likelihood, as well as other useful statistics. The other program transforms the original data by taking natural logarithms, computes the standard normal deviate and stores the results in a file for linear regression and plotting. A third program performs the latter two functions. The statistical computations were verified by processing the data with the well-known SAS procedures. The assumption of lognormality is demonstrated as valid for the study site and time series. Two other microcomputer programs are presented to compute the cumulative concentration probability distributions: 1) an approximate model based on the first two moments of a lognormal distribution, and 2) an exact, numerical method based on gaussian quadrature. Both models assume that flows and pollutant concentrations are jointly lognormally distributed (for point, upstream and non-point sources). A sixth microcomputer program plots the cumulative frequency distributions. Other utility programs were developed to dump data files to the screen of a microprocessor or to the paper printer.

The next step in the hierarchical procedure is application of a deterministic, continuous steady-state receiving water quality model. Instructions for data preparation are provided in the form of a user's manual. Although data requirements are intensive, the model is relatively inexpensive to execute when compared to continuous simulation of receiving water quality transients. The results are compared, as stated above, for the study site and time series. At the highest level of analysis, both frequency and duration of water quality violations are predicted, using a model developed in an earlier study of hydrologic and water quality modeling for instream flow strategies.
Toxics standards exist in North Carolina for about 40 parameters, but only five (mercury, copper, total chromium, nickel and zinc) are listed in the Division of Environmental Management (DEM) monthly monitoring report for upstream samples and point source influent samples. The Archie Elledge Wastewater Treatment Plant (Winston-Salem, N.C.) routinely measures and reports copper, total chromium, zinc and nickel concentrations in effluent and influent samples. However, none of these toxics concentrations were reported (and were presumably not measured) for upstream samples during the period of study, nor for several years before that. Although data are available from the Winston-Salem NURP study for stormwater and point source effluent concentrations to Salem Creek, a meaningful analysis is not possible without knowledge of the stream toxics background concentrations. Yet, the statistical procedures could be used effectively to determine the frequency with which toxics concentrations lethal to the fishery resource are reached. The deterministic models should eventually be extended to account for toxics, but adequate calibration and verification will be difficult without a supporting data base. A high degree of uncertainty still exists with regard to synergism, reaction rates, etc.

As noted above, the integrated methodology was applied to a river reach of Salem Creek, tributary to Muddy Creek, Yadkin-Pee Dee River Basin, North Carolina. The reach is currently classified as Class C (suitable for fish and wildlife propagation) by the State of North Carolina. The dissolved oxygen (DO) standard is 5.0 mg/l and the fecal coliform standard is a most probable number (MPN) of 1000 per 100 ml, the latter based on a logarithmic mean (five consecutive samples during any one month). The river reach has had historically high coliform bacteria levels, particularly during storm events. Using the statistically-based models with measured data for the time period November 1980 to August 1981, the fecal coliform standard is exceeded about 48 percent of the time during dry weather conditions. It should be noted that the fecal coliform bacteria concentrations measured from both upstream and effluent samples ranged in magnitude from 1 MPN/100 ml to 122,520 MPN/100 ml, resulting in high coefficients of variation. Frequency distributions are also presented for biochemical oxygen demand (BOD) and chemical oxygen demand (COD). The steady-state deterministic model produces cumulative frequency curves for the BOD-DO, cause/effect (coupled) reaction. It has been noted that this model is conservative (overpredicts impact) in the range of DO concentrations up to about the magnitude of the stream standard. At the most sophisticated level of analysis, the predictions are that the DO standard is equalled or exceeded 78 percent of the time on an hourly accounting basis; however, the same standard is violated 57 percent of the time during storm flow conditions in Salem Creek. Based on dissolved oxygen criteria for freshwater fish, species such as largemouth bass would have survived (no fish kills were reported during 1981 due to natural hydrologic phenomena). Periods of depressed dissolved oxygen levels (below 3.0 mg/l) were short in duration (2 to 3 hours at a time). Whether avoidance reactions were taken by fish or not is unknown, but certainly possible: there were 8 occurrences during which
the stream standard was violated for 12 consecutive hours, 3 occurrences each of durations of violations for 20 and 24 hour periods, and single occurrences of durations as lengthy as 48, 52 and 60 hours.

The hierarchical approach allows the user considerable flexibility. The simpler statistically-based microcomputer models are very inexpensive once the investment has been made in a microprocessor, and the configuration is highly portable. Both conventional and toxic pollutants can be analyzed and the output is in terms of the probability of occurrence of events of various magnitudes. These models cannot predict cause/effect, coupled reactions. They are limited in that the sensitivity of receiving water quality to proposed improvements to the existing configuration of unit processes, or the implementation of best management practices for nonpoint source control, cannot be readily investigated. Data requirements are very modest; nevertheless, greater confidence in the predictions is expected with longer time series. In essence, these models are excellent for use at the preliminary screening, planning level. When the need arises for more detailed information on system response to excitation, the deterministic long-term continuous simulation models may be applied. Even at the intermediate, steady-state level of analysis data requirements are substantial; however, execution costs are relatively modest. To obtain both cumulative frequency and duration of violations, simulation of water quality transients is required. The costs of long-term continuous simulation with relatively short time-steps (required to derive transient water quality and duration predictions) are high and an alternative when possible is attractive. Data requirements are substantial and will require careful coordination between data collection personnel and model users. The models presented in this study depend on data which is routinely being collected by various federal, state, or regional agencies. Unfortunately, the density of the gaging/sampling network is often inadequate and concurrent time series for all the important variables (rainfall, streamflow, receiving water quality, etc.) are difficult to find in the less-populated areas. Even though the results of simulation are presented in a very convenient format, biological damage is difficult to quantify. The expectations of inexperienced modelers or model users are quite often unreasonably high with regard to the accuracy of model predictions and the transferability of results to other sites. A deliberate hierarchical approach, from simple statistical analysis to application of the more complex computer models, appears to be the most sensible means of obtaining the best results as a function of available resources.
CHAPTER I

INTRODUCTION AND OVERVIEW OF
INTEGRATED METHODOLOGY

1.1 INTRODUCTION

On June 6, 1978 the President of the United States delivered a message to the Congress [House Doc. 95-347] in which he expressed he was "...particularly concerned about the need to improve the protection of instream flows...," and issued a directive to the Chairman and Members of the Water Resources Council on July 12, 1978 to prepare a report on the steps taken "...to develop effective operation and management techniques for protecting instream uses..." (Smith, 1979). The Instream-Flows Working Group, formed to implement the President's directive, classified problems in instream-flow into four categories: (1) inadequate quantity, (2) inadequate quality, (3) physical barriers, and (4) flow fluctuations (Instream-Flow Task Force, 1979). Inadequate quantity has an influence on water quality by adversely affecting waste assimilation. Therefore, the two aspects cannot be separated realistically. Water quality problems are the result of both point and nonpoint pollution sources, inadequate design of reservoir outlets, algal blooms in reservoirs, and others. Physical barriers (e.g., dams, weirs) interfere with migratory fish and their reproductive cycle. Flow fluctuation problems involve reservoir regulation.

Instream flow needs usually refer to amounts of flow required for traditional beneficial uses of water such as: navigation, hydropower generation, waste load assimilation (water quality), fish and wildlife (water quantity and quality), recreation (water quantity and quality) and consumptive uses (e.g., vegetation). The most desirable flow requirement would be that which satisfies several uses at once; however, a particular use is often defined as the most critical. With federal approval, states have classified stream segments as to their desired use and both water quality standards and effluent (discharge) standards are intimately related to such intended uses.

Instream flow assessments have traditionally resulted in the recommendation of a threshold value for the fishery resource: a minimum flow, usually determined from analysis of hydrologic records. This approach relies on the erroneous assumption that only flows below this "instantaneous minimum" will be detrimental to the fish (Smith, December 1979). The IFG incremental methodology (IFGIM) attempts to quantify the amount of potential habitat available for each life history state of a species as a function of streamflow. The IFGIM is intended to be used only where the flow regime is the dominant determinant of
the quality of the instream fishery or recreation resource and where hydraulic conditions are compatible with the theoretical basis of the models (i.e., steady flow within a rigid boundary). This method is composed of four basic components: (1) field measurement of stream channel characteristics using a multiple transect approach; (2) hydraulic simulation to determine the spatial distribution of combinations of depths and velocities with respect to substrate (bed material) and cover objects under alternative flow regimes; (3) application of habitat suitability criteria to determine weighting factors; and (4) calculation of weighted usable area (gross habitat index) for the simulated stream flows based on physical characteristics of the stream. The latter procedure roughly equates the total surface area of the simulated reach to an equivalent area of optimal (preferred) habitat. Weighted usable area (WUA) can be displayed as a function of streamflow for each life history state of the target species, as shown in Figure I-1 for smallmouth bass at a particular study site. From streamflow records, WUA may be presented as a function of mean monthly flow rates to facilitate comparison of changes in habitat potential between average and drought year conditions (see Figure I-2).

Four primary variables can be identified which determine the character of instream habitat conditions: (1) water chemistry; (2) food web relations; (3) flow regime; and (4) channel structure. Associated with each of these major variables are the respective subsets of variables which interact to provide the myriad of physical-chemical conditions to which the stream biota respond. Interactions among these represent the challenge in the difficult task of quantifying the effects of land and water management decisions on instream fishery resources.

Again, the IFGIM is very useful once it has been determined that flow regime is the dominant variable (assuming also that steady flow is compatible with streamflow conditions). Standard surveying and stream measuring techniques are used to obtain calibration data for IFG hydraulic simulation models. Transects are placed to characterize both hydraulic and instream resource (fishery habitat) conditions. Detailed information is obtained on the stream channel geometry and hydraulic conditions using a multiple transect approach for microhabitat description. The habitat suitability curves used in conjunction with the IFG methodology are based on the understanding that individuals of a species tend to select the most favorable conditions available within a stream for habitation, but will use less favorable conditions with less frequency, eventually leaving an area if possible before conditions become lethal. Subsequently, individuals would be most frequently observed (sampled) in nature inhabiting their most preferred habitat conditions. Implicit in the use of these criteria is the assumption that frequency of observation is, in fact, indicative of habitat preference and the understanding that the data base used to construct the curves was obtained in an unbiased manner. These criteria
Figure I-1. Weighted Usable Area Versus Discharge for Smallmouth Bass (Smith, 1979).
Figure I-2. Monthly Weighted Usable Area Values for Adult Smallmouth Bass Under Median and Drought Year Flow Conditions (Smith, 1979)
were prepared by life history stage for those streamflow parameters directly influenced by changes in flow regime or channel geometry and which were considered to most directly affect fish distribution: depth, velocity, substrate and temperature.

The IFG framework was developed for relatively pristine western streams and does not address the impact of water quality fluctuations. The IFGIM has been applied to some eastern streams, for example: the Greenbrier, Meadow and New Rivers of West Virginia (Joy, et al., 1981) and the Little Wabash River in Illinois (Herricks, et al., 1980). Neither study evaluated water quality conditions, which may not have been limiting factors. However, the need for incorporating water quality into the analysis procedure for stream resource flow requirements was recognized earlier by Grenney, Porcella and Cleave in an assessment of existing methodologies, prepared for the Fish and Wildlife Service (edited by Stalnaker and Arnette, 1976). A spectrum of water quality methodologies was recommended by a panel led by Mar in a workshop devoted to instream flow habitat criteria and modeling (edited by Smith, 1979). Grenney, Porcella and Cleave also identified dissolved oxygen (DO) as probably the single most important water quality parameter in fisheries management. Stream use classifications (including fish propagation) and corresponding water quality standards for the study site are presented in Chapter IV.

Water quality fluctuations in a river reach are due to both variations in pollutant loadings (from point and nonpoint sources) and hydrologic inputs (rainfall, streamflow). Continuous hydrologic and water quality simulation is proposed to derive frequency and duration of water quality violations in a stream reach, as a means of determining: the adequacy of existing or proposed flow conditions and levels of water pollution control upstream to support the intended uses of water in the selected segment.

1.2 OVERVIEW OF INTEGRATED METHODOLOGY

The use of frequency analysis in hydrologic and water quality modeling is treated in detail in Chapter II. The central theme is the prediction of frequency of occurrence of events of various magnitudes, both in terms of hydraulic and water quality variables. Hydrologic frequency curves (e.g., flow-duration curves) may be derived by continuous simulation of the rainfall-runoff process or by statistical evaluation of long-term hydrologic records. The derivation of cumulative water quality frequency curves and frequency distributions of duration requires continuous simulation of both quantity and quality transients. Cumulative frequency curves for pollutant concentrations in the receiving stream may also be obtained by statistically-based microcomputer programs. The mathematical basis for each of these models is presented in Chapter III, model applications and results in Chapter V, operation of models and instructions for data preparation in Chapter VI, and corresponding source program listings and data sets in the various appendices to this report. Several models are applied in this study which are documented in a pre-
vious report: Hydrologic and Water Quality Modeling For Instream Flow Strategies (Medina, 1982). These include: the STORAGE/TREATMENT AND RECEIVING WATER QUALITY FREQUENCY AND DURATION MODEL (STO/TRT RECEIVING); program RFREQ for rainfall frequency analysis; and program RATING which converts streamflow stage data to discharge by divided difference interpolation. The SYNOPTIC RAINFALL ANALYSIS PROGRAM (SYNOP), developed by Hydroscience for the U.S. Environmental Protection Agency (Areawide Assessment Procedures Manual, U.S. EPA, 1976), was modified to automatically define minimum interevent time by autocorrelation analysis. Other generalized computer programs applied to the study site were: the U.S. Geological Survey's DISTRIBUTED ROUTING RAINFALL-RUNOFF MODEL—VERSION II (DR3M) (Alley and Smith, 1982) and the U.S. Army Corps of Engineers' STORAGE, TREATMENT, OVERFLOW, RUNOFF MODEL (STORM) (Hydrologic Engineering Center, 1977). Three-dimensional color and black and white plots of water quality concentration in time and space were obtained by storing the results of STO/TRT RECEIVING simulation in files, then executing SAS/GRAPH: the computer graphics module of the Statistical Analysis System (SAS Institute, 1981). Several computerized data base systems were accessed: the North Carolina HYDROLOGIC INFORMATION STORAGE AND RETRIEVAL SYSTEM (HISARS) (Wiser, 1975); the U.S. Geological Survey NATIONAL WATER DATA STORAGE AND RETRIEVAL SYSTEM (WATSTORE) and the U.S. Environmental Protection Agency STORAGE AND RETRIEVAL SYSTEM (STORET) through the National Water Data Exchange (NAWDEX) (Edwards, 1977).

The model LEVEL III RECEIVING was originally developed by the principal investigator for the U.S. Environmental Protection Agency (Medina, 1979). It has been modified substantially for this study and new instructions for data preparation are provided in Chapter VI. Major improvements include: computation of the depth in the stream by divided difference interpolation from stage-discharge data and other curve-fitting techniques; print control commands that reduce the amount of output to an essential minimum; and the addition of BOD cumulative frequency curves that can now be viewed simultaneously with DO frequency curves.

Program RFREQ reads hourly precipitation from an NOAA rainfall tape and produces yearly frequency histograms as well as an average frequency histogram for the period of record. This procedure allows selection of the most representative year in the time series for more detailed autocorrelation analysis. Autocorrelation is used to define a minimum interevent time, as discussed in Chapter III. This value is required for a more general and complete storm event statistical analysis with SYNOP, presented for the study site in Chapter IV. The runoff time series is subsequently obtained by applying DR3M to the drainage basin: by a network of discrete overland flow and streamflow segments, for a given rainfall time series. Calibration and verification of the DR3M predictions is accomplished by comparison with field measured stage data, converted accurately to hydrographs by program RATING. STORM generates the pollutant loadings in the drainage basin. STO/TRT RECEIVING combines the point and nonpoint source pollutant loads and simulates mixing with receiving stream upstream loads to obtain water quality concentrations.
in time and space, cumulative water quality frequency curves and frequency distributions. Thus, the results may be interpreted in terms of the frequency of occurrence of water quality violations in a stream reach, as well as the duration of these violations. LEVEL III—RECEIVING is essentially the steady-state version of STO/TRT RECEIVING, without the detailed simulation capabilities of point source unit operations of the latter model, nor the capability of predicting the duration of water quality violations. The statistically-based microcomputer models and their utility programs also produce output in terms of the frequency with which a range of pollutant concentrations are exceeded in the stream: the central unifying theme of the complete, hierarchical modeling package. Violations are often defined by a minimum stream standard, but a range of standards may be investigated. The standards are set by state enforcement agencies in accordance with the stream use classification. An application to Salem Creek and Muddy Creek, Yadkin Pee-Dee River Basin, North Carolina is presented for a river reach classified suitable for fish and wildlife propagation.

The integrated approach towards accounting for water quality in determining instream flow strategies is summarized in Table I-1. The modeling package presented in this study satisfies levels II through IV. Level I is necessary regardless of the type of modeling package that may follow. It will ultimately be possible to store all of these models in a single microprocessor, due to advances in microelectronics, but at the higher levels of analysis execution times may still preclude practical operation of the computationally-intensive models in the near future.
TABLE I-1. Hierarchical Approach For Instream Water Quality Analysis

<table>
<thead>
<tr>
<th>Level I: Preliminary Screening</th>
</tr>
</thead>
<tbody>
<tr>
<td>Collection of maps, hydrologic and water quality historical data, stream intensive survey reports, routine computation of means and extremes, inventory of point sources, land use, identification of fish species in the river reach, etc.</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Level II: Statistically Based Frequency Analysis</th>
</tr>
</thead>
<tbody>
<tr>
<td>Identification of potential water quality impacts from derivation of pollutant cumulative frequency distributions, after analyzing historical data to determine if model assumptions (e.g., lognormality) are satisfied--for point, nonpoint and upstream flows and concentrations. This level requires application of small microcomputer programs and utility modules.</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Level III: Intermediate, Deterministically-Based Frequency Analysis</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steady-state, deterministic continuous simulation to derive cause/effect cumulative water quality frequency distributions, involving mainframe computer or mini-computer. A more refined frequency analysis, requiring substantially more data than Level II, with modest execution costs, with capability for producing time histories of system response to many combinations of waste inputs and levels of control.</td>
</tr>
</tbody>
</table>

LEVEL III - RECEIVING, other supporting computer models.
Level IV: Advanced, Deterministically-Based Frequency and Duration Analysis

Deterministic, continuous simulation of receiving water quality transients to derive both frequency and duration of water quality violations; three-dimensional computer graphics capability to view cause/effect (pollutant/water quality) relationships in time histories of system response to varying inputs and configurations of wastewater treatment unit operations, etc. At this level, it is likely that mainframe computer simulation will continue to be more convenient due to long execution times; data requirements are substantial; assessment of biological damage possible on the basis of combined frequency and duration of water quality violations of established standards for fishery resource.

STO/TRT RECEIVING, other supporting computer models.
CHAPTER II
FREQUENCY ANALYSIS IN
HYDROLOGIC AND WATER QUALITY
MODELING

2.1 HISTORICAL REVIEW

The random component of hydrologic signals requires that rational water resource management tools account for hydrologic uncertainty and associated water quality variability. The practice of performing frequency analysis on historical data collected from natural phenomena has been in existence for almost a century. Frequency analysis of streamflow data is believed to have been first applied to flood studies by Herschel and Freeman (Foster, 1934). Today, modern electronic computers are used to generate synthetic streamflows because in many cases existing records are not sufficiently extensive to provide estimates of important statistics. Such approximate models are sufficiently realistic to improve the planning process significantly (Fiering and Jackson, 1971). Simulated streamflow data have been obtained for most river basins in North Carolina (Wiser, 1981). Model parameters required for simulation are obtained by comparing predicted data with observed data when available. Precisely, the justification for continuous (long-term) simulation in dealing with receiving water quantity and quality is the probability of occurrence of hydrologic events of various magnitudes (Linsley and Crawford, 1974).

The conventional approach of selecting single design events during critical time periods (low-flow conditions) for water resource management is inadequate for several important reasons:

- No reliable probability or frequency of occurrence can be determined for the single event (Linsley and Crawford, 1974).

- The most critical impact on receiving water quality does not necessarily occur under low flow conditions, because of intermittent runoff pollutant shock loads (Heaney, et al., 1977).

- No accepted design event condition exists which also specifies a design antecedent dry-weather period (Heaney, et al., 1977).
Worst-case conditions of receiving water quality have been arbitrarily defined (e.g., 7-day, 10-year low flow) in conventional waste allocation studies. The use of continuous computer simulation to develop water quality frequency curves to screen alternative control strategies is becoming well established (e.g., Black and Veatch, 1974; Heaney, et al., 1977; Donigian and Linsley, 1979; Medina, 1979; Scholl, et al., 1980; Medina, et al., 1981; Medina and Buzun, 1981; Medina and Buzun, 1982). Continuous simulation produces results which can be interpreted for a wide range of water quality standards rather than a fixed mythical standard (e.g., 5 mg/l of DO), and a wide range of streamflow conditions. Extension of the methodology for instream flow strategies appears to offer particular advantages if frequency and duration of water quality violations can be more closely correlated to instream uses such as fishery management (Medina, 1982).

2.2 HYDROLOGIC FREQUENCY STUDIES

The traditional approach to the problem of determining theoretical probabilities of hydrologic events has been the derivation of frequency curves. These curves relate the magnitude of a variable to frequency of occurrence, and are an estimate of the cumulative distribution of the population of that variable as prepared from a sample of data (Riggs, 1968). The probability of a single event, say \( x_1 \), is defined as the relative number of occurrences of the event after a long series of trials or observations from a historical record:

\[
P(X = x_1) = \frac{n_1}{N}
\]  

where \( X \) denotes a hydrologic event, say streamflow

\( x \) = magnitude of that event

\( n_1 \) = number of occurrences of event of magnitude \( x_1 \)

\( N \) = total number of observations of event \( X \).

The number of occurrences \( n_1 \) is the frequency, whereas \( n_1/N \) is the relative frequency. When the number of values a random variable can take on is restricted to an integer number (say 0, 1, 2, ...), the random variable is called discrete and its probability law is usually presented in the form of a probability mass function (PMF):

\[
P_x(x_1) = P(X = x_1)
\]  

where \( P_x(x_1) \) is the probability mass function of \( X \).
and, by definition

\[ \sum_{\text{all } x_i} P_X(x_i) = 1 \]  

\[(2.3)\]

where \( x_i \) = discrete values of random variable \( X \).

Equation (2.2) describes the probability or frequency distribution of a random variable. An equivalent means is obtained through the use of the cumulative distribution function (CDF):

\[ F_X(x_i) = P(X \leq x_i) \]

\[ = \sum_{x_i \leq x} P_X(x_i) \]  

\[(2.4)\]

for discrete random variables, and the function increases monotonically from a lower limit of zero to an upper bound of unity.

Unlike the discrete variable, the continuous random variable is free to take on any value on the real axis. If the abscissa (x axis) is separated into a large number of short intervals \( \Delta x \), and the ordinate is the function \( f_X(x) \), such that the area under the curve in an interval represents the probability that the random variable will take on a value in that interval, then:

\[ P(x_1 < X < x_2) = \int_{x_1}^{x_2} f_X(x) \, dx \]  

\[(2.5)\]

where \( f_X(x) \geq 0 \)  

\[(2.6)\]

\[ \int_{-\infty}^{\infty} F_X(x) \, dx = 1 \]  

\[(2.7)\]

and

\[ f_X(x) = \lim_{\Delta x \to 0} \frac{\Delta F(x)}{\Delta x} = \frac{dF(x)}{dx} \]  

\[(2.8)\]

= probability density function (PDF).

The cumulative distribution function is defined in terms of the PDF as
\[ F_X(x) = P(X \leq x) = \int_{-\infty}^{x} f_X(u) \, du \] (2.9)

where \( u \) = dummy variable of integration.

The relationship between the PDF and CDF of a random variable is illustrated in Figure II-1.

The cumulative distribution function has been defined as the expected number of occurrences less than a given value; however, it is also convenient to examine its complement -- the expected number greater than or equal to the given magnitude:

\[ G_X(x) = 1 - F_X(x) = P(X \geq x) \] (2.10)

It should be noted again that for hydrologic applications the CDF or its complement may be referred to as frequency curves. Earlier statisticians also used the term cumulative frequency function (Burr, 1942). The area under either the CDF curve or its complement is meaningless: expected frequencies in any given range are found by simply taking the difference between ordinates. For example, \( P(x_1 < X \leq x_2) \) is evaluated as \( F_{X}(x_2) - F_{X}(x_1) \). The probability distribution of sampled data taken from a continuous distribution is a special case of discrete distributions and may be computed in the form of the arithmetic summations presented earlier (Benjamin and Cornell, 1970).

Hydrologic applications of frequency curves include: the design of bridge openings, channel capacities, flood-plain zoning, industrial and domestic water-supply systems, storage reservoirs, and forecasting problems (Riggs, 1968). The flow-frequency, or flow-duration, curve specifically accounts for hydrologic uncertainty in the design and planning of flood-control or drought-relief facilities. The duration curve is the integral of the probability curve, and early investigators concluded the latter to be described best by the Gauss-Laplace normal distribution curve (Beard, 1943). A typical flow-duration curve for a hypothetical watershed is shown in Figure II-2.

In later studies, an index of the variation of flow in a stream was developed from duration curves of discharge (Lane and Lei, 1950). An extensive treatise on flow-duration curves is available elsewhere (Searcy, 1959). These curves are considered useful even though the events may not be completely independent of each other; that is, they may be serially correlated (Riggs, 1968).
Figure II-1. The Probability Density Function and its Cumulative Distribution Function.

Figure II-2. Flow-Duration Curve for Hypothetical Watershed
Besides the obvious usefulness of frequency analysis in averting flood disaster, it is a means of achieving efficient designs for hydraulic structures. If a hydraulic structure is underdesigned through inadequate or inaccurate data or methods, the results may be regrettably catastrophic in terms of loss of property and lives. However, non-failure is often the result of overdesigned, very safe, but also very expensive structures. A truly efficient design will be achieved only if costs are related to risk and frequency analysis (Kite, 1977). An analogy can be drawn to the traditional approach of selecting single design events during presumed critical time periods (worst-case 7-day, 10-year low flow conditions in waste allocation studies), without consideration for optimality between costs and risks. The concept of water quality frequency curves and frequency distributions of duration are explored in the next section.

2.3 WATER QUALITY FREQUENCY CURVES

Figure II-3 illustrates water quality frequency curves for two levels of upstream water pollution control schemes, for a hypothetical watershed, in terms of an established receiving water quality standard. At the higher level of control, it is expected that a higher number of events equal or exceed the established water quality standard minimum concentration. Thus, fewer occurrences of water quality standard violations are predicted. Along with the frequency distribution of consecutive time periods of violation of a selected stream standard (see Figure II-4), cumulative water quality frequency curves form an integral part of the methodology proposed for instream flow strategies in Chapter III and demonstrated in Chapter V.

The percent of time equaled or exceeded for a given magnitude of the stream standard is computed from:

\[
\text{\% Time Equaled or Exceeded} = 100 \left[ \frac{N-n_i}{N} \right] \tag{2.11}
\]

where \( n_i \) = cumulative frequency of occurrence (successive partial sums) in class interval \( i \)

\( i = 1, 2, \ldots, I \)

\( I = \) number of class intervals. Frequencies of dissolved oxygen concentrations are computed in the receiving water quality model for class intervals of 0.5 mg/L, from 0.0 to 15.0 mg/L (i.e., 31 class intervals are defined).

In contrast to the century-old practice of frequency analysis for flood control, drought severity, and other quantitative hydrologic applications, its use in water quality control has developed within the
Figure II-3. Cumulative Water Quality Frequency Curves for Hypothetical Watershed

Figure II-4. Frequency Distribution of Consecutive Time Periods of Violation of the Selected Stream Standard
last decade. Downstream damages, in terms of water treatment costs at a point, have been related to probability of occurrence or exceedance (Kneese and Bower, 1968). The damages varied according to the dilution provided by streamflow. Cumulative frequency curves have been proposed to relate probability to annual, stream waste-assimilative capacity (Velz, 1970) under natural hydrologic variations. In a study by Hydrocomp International and Black & Veatch of the South Platte River (where the modeling area was centered around Denver, Colorado) minimum dissolved oxygen cumulative frequency curves were compared for various dry-weather wastewater treatment plant configurations (Denver Regional Council of Governments, 1974).

2.4 DETERMINISTIC SIMULATION VERSUS DERIVED DISTRIBUTION APPROACHES

A physical law describes the deterministic evolution of natural processes (e.g., mechanics of surface runoff), but a probabilistic interpretation is not necessarily due to ignorance of governing physical phenomena. The controversy of determinism and causality versus randomness and probability has been the topic of extensive discussions (Papoulis, 1965): the difference lies not in the nature of the phenomena, but in the quantities in which the observer is interested. If the outcome of one experiment is of interest, then the model might be deterministic, but with some uncertainty due to certain errors in certain ranges of the relevant parameters. The use of single events for design or water resources management has been discarded for several reasons stated earlier in Section 2.1. Long-term, continuous deterministic simulation essentially predicts the average system response to a large number of events because these models depend upon parameters obtained from measured data (statistical samples). Thus, long-term characterization of the rainfall-runoff process and derivation of both hydrologic and water quality frequency curves can be achieved by either deterministic (physically-based) simulation or probabilistic (derived distribution) methods.

Derived distribution approaches require that an assumption be made of a theoretical frequency distribution for the population of events, and the statistical parameters of the distribution must then be computed from the sample data. For example, from knowledge of hydrologic relationships, statistical distributions can be derived for storm event dependent variables such as surface runoff and overflow to receiving streams. This approach is highly dependent on how well the distributions of the original variables can be hypothesized (Loganathan and Delleur, 1982). Quite often these methods yield closed form solutions which are useful for preliminary water resource assessment.

Both approaches are data dependent, but the derived distribution approach is computationally less demanding. Of course, physically-based models are not completely deterministic because many model parameters (infiltration, reaction rates, etc.) are quite difficult to estimate.
for most practical applications. Loganathan and Delleur (1982) have
demonstrated there is strong evidence that hydrologic variables such as
surface runoff volume, duration and interevent time are exponentially
distributed. They proposed the lognormal distribution and the beta
distribution for the pollutant concentrations in the receiving stream.
Warn and Brew (1980) present an analytical method that assumes
log-normal distributions for river and point discharge flows. The method
is an approximate probability model of the mass balance equation based
upon the first two moments (mean, variance) of the downstream concentra-
tion. A two-parameter log-normal distribution is used and provides
accurate values if the assumption of log-normal flows is valid. The
method was extended to include stormwater flows and concentrations
(DiToro, 1982), provided that these flows and concentrations and the
upstream flows and concentrations can be assumed to be independent.
Earlier work by DiToro (1980) established the adequacy of probabilistic
analysis of the response of one-dimensional advective-dispersive
systems. In a survey of stochastic models, Padgett (1980) reviews the
application of random differential equation approaches to the computa-
tion of the probability distributions of BOD and DO in streams.

The statistical computer models presented in this study are based
on assumptions of lognormally-distributed flows and concentrations for
nonpoint sources, point sources and upstream sources. Recently
completed studies under the National Urban Runoff Program (NURP) support
this assumption (Athayde, 1983). Figures 11-5 to 11-12 are normal pro-
bability plots of the natural logarithms of streamflow, surface runoff
and pollutant concentrations in stormwater for all the nationwide sites
under the NURP. The analyses were performed using the UNIVARIATE pro-
cedure of the Statistical Analysis System (SAS), SAS Institute, at the
National Computation Center facilities. These plots represent at least
1500 data points, and obviously appear linear in nature. Similar plots
are presented for the study site, near Winston-Salem, North Carolina,
in Chapter IV. Linear regression was computed on all the relevant var-
iables for the study site, and the results are presented in Chapter V:
the validity of the log-normal distribution assumptions is clearly
demonstrated by extremely high correlation coefficients in all cases.
Figure II-5. Normal Probability Plots of Logarithms of Upstream Flows and Stormwater Surface Runoff

Figure II-6. Normal Probability Plots of Logarithms of Stormwater Organic Pollutant Concentrations
Figure 11-7. Normal Probability Plot of Logarithms of Stormwater Fecal Coliform Concentrations

Figure 11-8. Normal Probability Plot of Logarithms of Stormwater Total Suspended Solids Concentrations
Figure II-9. Normal Probability Plots of Logarithms of Stormwater Nitrogen Concentrations

Figure II-10. Normal Probability Plots of Logarithms of Stormwater Phosphorus Concentrations
Figure II-11. Normal Probability Plot of Logarithms of Stormwater Lead Concentrations

Figure II-12. Normal Probability Plots of Logarithms of Stormwater Toxics Concentrations, Copper and Zinc
CHAPTER III

METHODOLOGY

This chapter is devoted primarily to description of the mathematical foundation of models developed or modified for use in this study: for example, the updated version of LEVEL III - RECEIVING and the statistically-based receiving water quality frequency models. The features of other programs developed and documented elsewhere (e.g., HISARS, STORM, DR, M) are briefly summarized and their mathematical basis is discussed where appropriate. The STORAGE/TREATMENT AND RECEIVING WATER QUALITY FREQUENCY AND DURATION MODEL (STO/TRT RECEIVING), program RATING which converts streamflow stage data to discharge by divided difference interpolation, the SYNOPTIC RAINFALL DATA ANALYSIS PROGRAM (SYNOP), and program RFREQ (which supplements SYNOP by producing yearly frequency histogram plots from NOAA rainfall data) are treated in detail in a previous study (Medina, 1982). An analysis of the hourly rainfall time series from 1948 to 1975 (recorded near Winston-Salem, North Carolina at a first-order weather station) is presented in Chapter IV as an integral part of the description of the study area and its climatology. Other model applications to Salem Creek and Muddy Creek (Yadkin-Pee Dee River Basin) are presented and results interpreted in Chapter V. Operation of models, pertinent input data instructions and programming considerations are presented in Chapter VI.

3.1 RAINFALL TIME SERIES ANALYSIS

An integral part of the assessment of storm-derived pollutant loads on receiving water quality is the statistical evaluation of rainfall records. The purpose of SYNOP is to summarize the variables of interest (volume, duration, intensity and time between storm events) and statistically characterize the rainfall record to determine seasonal trends (Areawide Assessment Procedures Manual, U.S. EPA, 1976). The hourly rainfall data are summarized by storm events, each with an associated unit volume, duration, average intensity and time since the preceding storm (measured from the midpoint of the successive storms). Thus, a storm definition must be established to determine when in the hourly record a storm begins and ends. Program SYNOP delineates storm events as rainfall periods separated by a fixed minimum number of consecutive hours without rainfall (user-defined). To avoid an arbitrary definition of independence, the program has been modified to statistically derive a minimum interevent time (MIT) on the basis of autocorrelation analysis of the hourly rainfall of a representative year in the time series (Medina, 1982).

For hydrologic processes, it is practical to estimate the autocorrelation coefficients by an open-series approach (Yevjevich, 1972 and Fiering and Jackson, 1971):
\[ r_I(k) = \frac{\sum_{i=1}^{n-k} x_i x_{i+k} - \frac{1}{n-k} \left[ \sum_{i=1}^{n-k} x_i \right] \left[ \sum_{i=k+1}^{n} x_i \right]}{\left[ \sum_{i=1}^{n-k} x_i^2 - \frac{1}{n-k} \left( \sum_{i=1}^{n-k} x_i \right)^2 \right]^{0.5}} \]  

(3.1)

where \( r_I(k) \) = sample estimate of lag-k autocorrelation coefficient for hydrologic process I,

\[ x_i = \text{discrete data series (observations) of hydrologic process I, for } i = 1, 2, \ldots, n, \]

\[ n = \text{total number of data points or observations,} \]

\[ k = \text{number of hourly lags.} \]

The tolerance limits for a normal random time series which is circular and of lag 1, \( TL[r_I(1)] \), are given by (Anderson, 1942):

\[ TL[r_I(1)] = \frac{-1 \pm t_{\alpha} \sqrt{n-2}}{n-1} \]  

(3.2)

where \( t_{\alpha} \) = standardized normal variate corresponding to probability level \( (1 - \alpha) \).

A circular time series is defined as a series where the last value is followed by the first so that the time series repeats itself. Equation Equation (3.2) has been extended for use with an open series, for the general lag case (Yevjevich, 1972). At a 95 percent probability level, the tolerance limits are given by:

\[ TL[r_I(k)] = \frac{-1 \pm 1.645 \sqrt{n-k-1}}{n-k} \]  

(3.3)

A plot of the serial correlation coefficients, \( r(k) \), against the number of lags, \( k \), is called a correlogram. The technique of autocorrelation analysis is essentially a study of the behavior of the correlogram of the process under investigation (Quimpo, 1968). The model compares the value of \( r(k) \) obtained from equation (3.1) with \( TL[r_I(k)] \), computed by equation (3.3), for the corresponding number of hourly lags \( k \). The minimum interevent time (MIT) which separates...
independent wet-weather events is defined as the minimum value for k for which \( r(k) \) is not significantly different from zero at a 95 percent probability level.

Once the MIT has been defined, the storms are separated accordingly and the statistics of the storm parameters are then computed. The mean standard deviation and coefficient of variation (standard deviation / mean) are determined for storm intensity, duration, unit volume and time between storms. If storm intensities and durations are independent, the mean storm volume will equal the product of mean storm intensity and mean duration. However, in many areas and during certain seasons they are not independent: for example, long less-intense storms tend to occur in the winter, and short high-intensity storms tend to occur in the summer. To avoid this potential error, the rainfall analysis program determines the mean unit volume from the individual storm volumes.

It should be noted that in North Carolina intense rainstorms occur in steep mountain terrain (orographic precipitation), especially in the southern portion. In the Piedmont region (location of study site, see Chapter IV) the seasonal (summer, winter) behavior described above is clearly observable in monthly summaries and plots of average intensity and duration. A particular advantage of this type of analysis is that if a particular season or period is considered critical due to adverse receiving water characteristics or greater pollutant accumulation rates, the representative summary may simply be made on the long term record of storms occurring during the selected season.

The frequency distribution of a random variable was defined in Chapter II and an example was provided. It was stated earlier that autocorrelation analysis of the hourly rainfall of a representative year in the time series led to the definition of the minimum interevent time. The program RFREQ was developed to produce a frequency histogram of hourly rainfall for each year of the entire time series subjected to analysis by SYNOP, plus an average frequency histogram of hourly rainfall for the entire time series. Thus the most representative year may be selected for detailed processing by equations (3.1) and (3.3) at a savings in computer time.
3.2 HYDROLOGIC MODELING

Problems in instream flows have been classified into four categories: (1) inadequate quantity, (2) inadequate quality, (3) physical barriers, and (4) flow fluctuations. Inadequate quantity of flow has an influence on water quality by adversely affecting waste assimilation. A comprehensive watershed model is required that: faithfully represents the physical system; can process the rainfall time series, account for all the hydrologic abstractions, and predict surface runoff in time and space as a residual of the gross precipitation input.

**Hydraulic Simulation**

Hydraulic simulation for instream flow studies is defined as the description of the changes in distribution of velocities, depths and substrates as a function of discharge (Bovee and Milhous, 1978). Depth and velocity of flow are, of course, a function of channel geometry also. An accurate representation of velocity and depth of flow is governed by the St. Venant equations, respectively, the dynamic and continuity equations for gradually varied, unsteady flow:

\[
\frac{\partial y}{\partial x} + \frac{V}{g} \frac{\partial V}{\partial x} + \frac{1}{g} \frac{\partial V}{\partial t} = S_o - S_f
\]

\[
A \frac{\partial V}{\partial x} + V \frac{\partial A}{\partial x} + \frac{\partial A}{\partial t} = 0
\]

where

- \( y \) = depth of flow
- \( V \) = velocity of flow
- \( x \) = longitudinal distance
- \( t \) = time
- \( g \) = gravitational acceleration constant
- \( S_o \) = invert slope
- \( S_f \) = friction slope, and
- \( A \) = flow area.

However, numerical solutions of these equations are costly in terms of computer time. Most modern hydrologic simulation models adopt a kinematic wave approach in which disturbances are allowed to propagate only in the downstream direction. As a consequence, downstream conditions do not affect upstream computations.
For both overland flow and open channel flow segments, lateral inflow must be considered. The overland flow segments receive lateral inflow in the form of excess precipitation. Thus, equations (3.4) and (3.5) become

\[
\frac{3y}{3x} + \frac{V}{g} \frac{3V}{3x} + \frac{1}{g} \frac{3V}{3t} = (i - f + 2q_L/b) \frac{V}{gy} + S_0 - S_f
\]  

(3.6)

\[
\frac{3q}{3x} + \frac{3y}{3t} = (i - f) + 2q_L/b
\]  

(3.7)

where \( q \) = discharge per unit width of channel, say cfs/ft  
\( q_L \) = lateral inflow, say cfs/ft  
\( i \) = rainfall intensity, volume per unit time per unit area, say ft/sec  
\( f \) = infiltration rate, volume per unit time per unit area, say ft/sec, and  
\( b \) = width of free surface, say ft ;

for assumptions of a moderately wide rectangular channel \((y/b<1)\), small bottom slope, and uniform velocity distribution. The kinematic wave approach maintains the continuity equation as above, but the momentum equation is replaced by a stage-discharge relation based on either the Chezy or Manning friction formula

\[ q = \alpha y^m \]  

(3.8)

where, typically,

(1) for laminar flow,

\[ \alpha = \frac{g S_o}{2\nu} \text{ and } m = 3 \]  

(3.9)

where \( \nu \) = kinematic viscosity, and

(2) for turbulent flow,

\[ \alpha = \frac{1.49}{n} \sqrt{S_o} \text{ and } m = \frac{5}{3} \]  

(3.10)
Thus $a$ and $m$, the kinematic wave model parameters, are related to the roughness and geometry of the basin and must be determined accordingly. Since disturbances can only propagate downstream, numerical solutions are simpler while still retaining some of the nonlinear effects of the dynamic equation. The solution to overland-flow problems simplifies to the continuity and momentum equations with $q_L = 0$, respectively:

$$\frac{\partial v}{\partial t} + \frac{\partial q}{\partial x} = i - f$$  \hspace{1cm} (3.11)

$$q = ay^m$$  \hspace{1cm} (3.12)

Any wavelike behavior must enter through the continuity equation and the approach precludes changes in surface profile due to dynamic variations. The corresponding set of equations for stream segments, or channelized flows, to which the only significant inputs are continuous along the stream axis and consist of rainfall, infiltration, and overland flow is:

$$\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = q_i$$  \hspace{1cm} (3.13)

$$Q = a_s A^m$$  \hspace{1cm} (3.14)

where

$Q =$ discharge rate, say cfs
$A =$ flow cross-sectional area, say ft. $^2$
$q_i =$ continuous lateral inflow rate of overland flow, say cfs/ft
$a_s, m_s =$ kinematic parameters for stream segments.

Typically, the value of $m = 3/2$ for small streams. In a stream segment, equation (3.14) may be replaced by the classical Manning relationship

$$Q = \frac{1.49}{n} R^{2/3} A \sqrt{S_0}$$  \hspace{1cm} (3.15)

where $R =$ hydraulic mean radius.

Routing through reservoirs is commonly accomplished by application of the continuity equation (modified-Puls method), in difference form given by:
\[ Q_{t+\Delta t} + \frac{2 \psi_{t+\Delta t}}{\Delta t} = I_t + I_{t+\Delta t} + \frac{2 \psi_t}{\Delta t} - Q_t \] (3.16)

where \( Q_t, Q_{t+\Delta t} \) = outflow rates at times \( t \) and \( t+\Delta t \), cfs

\( I_t, I_{t+\Delta t} \) = inflow rates at times \( t \) and \( t+\Delta t \), cfs

\( \psi_t, \psi_{t+\Delta t} \) = storage volumes at \( t \) and \( t+\Delta t \), ft\(^3\)

\( \Delta t \) = time step used in the kinematic wave model.

Distributed Routing Rainfall-Runoff Model

The U.S. Geological Survey's Distributed Routing Rainfall-Runoff Model-Version II (DR\(_M\)) provides detailed simulation of storm-runoff periods selected by the user and a daily soil-moisture accounting between storms, using rainfall and data describing the physical characteristics of the drainage basin as input (Alley and Smith, 1982). A drainage basin is essentially represented as a set of overland-flow, channel and reservoir segments. Kinematic wave theory is used for routing flows over contributing overland-flow areas and through the channel network, as described mathematically in the preceding section. A detailed representation of the Salem Creek and Muddy Creek drainage basins near Winston-Salem, North Carolina is illustrated in Chapter V. All model features are well documented in the user's manual; however, it is appropriate to comment on the numerical solution technique to the kinematic wave equations found to be most practical for long-term simulation (year or longer) of a large drainage basin (64 square miles; 16,576 hectares).

Prior to release of the new version, the 1978 DR\(_M\) provided only an explicit finite-difference scheme for solution of the kinematic wave routing in channels and overland flow segments. Although correct solutions were obtainable for the smaller sub-basins, calibration of the model for the aggregate network representing the entire basin was never accomplished. Such difficulties were not encountered by choosing the method of characteristics option in the newer version of DR\(_M\). The implicit finite-difference scheme was not chosen because of computer time considerations. An exhaustive investigation of the causes of the problems (choice of \( \Delta t \), segmentation, etc.) was not possible due to time limitations.

Combining equations (3.13) and (3.14) and dropping subscripts for convenience yields:
\[
\frac{\partial A}{\partial t} + \alpha m A^{m-1} \frac{\partial A}{\partial x} = q
\]  
(3.17)

and its solution provides values of area \(A\) that can be converted to discharge using equation (3.14) (Alley and Smith, 1982). Equation (3.17) can be represented by the characteristic equations below (Eagleson, 1970):

\[
\frac{dx}{dt} = \alpha m A^{m-1}
\]  
(3.18)

\[
\frac{dA}{dt} = q
\]  
(3.19)

Integration of equations (3.18) and (3.19) can be performed explicitly if the lateral inflow, \(q\), is assumed uniform in time and space. For a given model segment, \(q\) is indeed spatially constant and piecewise constant in time. Thus, the assumption is valid by integrating over time steps where \(q\) remains constant. The equations actually solved by DR3M are:

\[
\Delta x = \frac{\alpha}{q} [ (q \Delta t + A(x,t))^m - A(x,t)^m ]
\]  
(3.20)

\[
A(x+\Delta x, t+\Delta t) = A(x,t) + q \Delta t
\]  
(3.21)

for \(q \neq 0\), and

\[
\Delta x = \alpha m A(x,t)^{m-1} \Delta t
\]  
(3.22)

\[
A(x+\Delta x, t+\Delta t) = A(x,t)
\]  
(3.23)

for \(q = 0\).

Equations (3.20) and (3.22) are used to follow the characteristic paths in the \(x-t\) plane. The flow area is determined at points along the characteristic paths by equations (3.21) and (3.23).

**Rating Table Interpolation**

Any hydrologic model must be calibrated and verified with actual field-measured data due to the uncertainty introduced by the mathematical abstraction of the physical system, unknown magnitude of certain co-
Coefficients, etc. Hourly stream gage height records were available from the U.S. Geological Survey for the study site as well as stage-discharge rating tables updated regularly. Program RATING was written to convert gage height to discharge by an accurate interpolating scheme, with a known tolerance (Medina, 1982). Based on a divided-difference table computed by using the Newton form for the interpolating polynomial (Conte and de Boor, 1972) numerical error is minimized:

\[
P_{k+1}(x) = \sum_{i=0}^{k+1} f[x_i, \ldots, x_{k+1}] \prod_{j=i+1}^{k+1} (x - x_j) \quad (3.24)
\]

where \(x_0, x_1, x_2, \ldots\) = given distinct points

\(f(x_0), f(x_1), f(x_2) \ldots\) = values of a function \(f(x)\) at these points for

\(k = 0, 1, 2, \ldots\), until satisfied on the basis of an error tolerance.

The subroutine generating the divided difference table is used in both of the deterministic water quality simulation models, LEVEL III - RECEIVING and STO/TRT RECEIVING, to obtain stream depth from known discharge and known stage-discharge power relationships.

3.3 DETERMINISTIC WATER QUALITY MODELING

Rainfall on an impervious area must first wet the surface and depression storage must be filled before any stormwater runoff is generated. The initial amounts of rain begin to dissolve the available water soluble pollutants. As rainfall continues, surface runoff develops and carries dissolved material with it in both the overland flow and channel flow phases. With increased flow and velocity, the suspended solids fraction is carried off the watershed (pollutant washoff). In the pervious fraction of a drainage basin, stormwater runoff and associated pollutant removal are similar but additional rainfall is lost to infiltration. Thus, even though pollutant concentrations may be just as great at given discharges, fewer total pollutants are removed. Of course, not all the available pollutants are removed during a storm event. The percentage removed depends on constituent properties, the land surface (type of cover), the rainfall intensity (rainfall splash detaches soil particles), and particularly the stormwater volume flow rate (Overton and Meadows, 1976). The watershed pollutant washoff model is followed by the receiving water quality model which usually predicts concentrations in time and space. Degradation of stream water quality must then be evaluated in terms of degree of impairment of its beneficial uses for either instream or off-stream purposes.
Computation of Surface Runoff Quality

In the computation of surface runoff quality a number of assumptions are generally made: (1) the amount of pollutant which can be removed during a storm event is dependent on rainfall duration and initial quantity of pollutant available for removal; (2) no pollutants decay due to chemical changes or biological degradation during the runoff process, and (3) the amounts of pollutants percolating into the soil by infiltration are not significant. The first assumption can be refined for mathematical derivation such that the rate of removal of pollutants by surface runoff is proportional to the amount of pollutant remaining, and to the runoff intensity. The process can be modeled by a first-order differential equation:

\[ \frac{dP}{dt} = kP \]  

(3.25)

which integrates to

\[ P_o - P = P_o \left(1 - e^{-kt}\right) \]  

(3.26)

where 

- \( P_o \) = pollutant originally on ground, mg
- \( P \) = pollutant after time \( t \), mg
- \( k \) = constant, and is assumed to be directly proportional to the rate of runoff,
- \( b \) = runoff intensity
- \( b \) = constant.

For each time step, the runoff rate is determined from the hydrograph and a value of \( P \), which becomes the new value of \( P_o \) for the next time step, is computed.

The watershed pollutant washoff model chosen was STORM because of ease of data preparation, ease of linkage with an hourly receiving water quality model, and applicability to drainage basins with urban and non-urban fractions and multiple land uses (e.g., residential, commercial, industrial, open space and rural, pastures, farming, and forests). Since a significant portion of the pollutants in the study site come from non-urban land uses (see Chapter IV), the daily pollutant accumulation method was used (Hydrologic Engineering Center, 1977):

\[ P_p = \sum_{i=1}^{L} \left(F_{pi} \cdot (A \cdot P_{T_i}) \cdot N_D\right) + P_{po} \]  

(3.27)
where \( P_p \) = pollutant p at beginning of storm, total pounds
\( F_{pi} \) = accumulation of pollutant p on land use i, lbs/acre/day
\( A \) = total area in basin, acres
\( PT_i \) = percent of basin in land use i
\( N_D \) = number of days without runoff since last storm
\( P_{po} \) = pollutant p remaining at end of the last storm, total pounds,
\( L \) = number of land use types.

Accurate land use types and percentages were obtained from the Land Resources Information Service, State of North Carolina, for the Salem Creek basin, discussed further in Chapter IV.

**Pollutant Transport Systems**

The unifying principle of conservation of mass may be applied to each subsystem of the urban and non-urban (natural) environments to describe the transport of pollutants. Figure III-1 represents a generalized component of the physical system to be modeled, which may characterize: (1) a pipe segment of the sanitary sewer system, (2) a storage/treatment unit (e.g., primary clarifier in the municipal wastewater treatment plant), or (3) a reach of the receiving body of water. In essence, each of these subsystems provides engineered or natural storage/treatment; therefore, all of these subsystems may be approximated by the one-dimensional version of the classical convective-dispersion equation,

\[
\frac{3C}{3t} = \frac{3}{3x} \left[ E \frac{3C}{3x} - UC \right] + \Sigma S_i
\]

where \( C \) = concentration of water quality parameter (pollutant), \( M/L^3 \),
\( t \) = time, T,
\(-E\frac{3C}{3x}\) = mass flux due to longitudinal dispersion along the flow axis, the \( x \) direction, \( M/L^2T \),
\( UC \) = mass flux due to advection by the fluid containing the mass of pollutant, \( M/L^2T \),
\( S_i \) = sources or sinks of the substance \( C \), \( M/L^3T \),
\( i = 1, 2, \ldots, n \),
\( n \) = number of sources or sinks,
\( U \) = flow velocity, \( L/T \), and
\( E \) = longitudinal dispersion coefficient, \( L^2/T \).
FLOW BALANCE:

\[ \frac{dV}{dt} = Q_1(t) - Q_2(t) \]

MASS BALANCE:

\[ \frac{\partial C}{\partial t} = \frac{\partial}{\partial x} \left[ E \frac{\partial C}{\partial x} - UC \right] + \sum S_i \]

Figure III-1. A Generalized Component of the Physical System
The source/sink term accounts for biochemical processes (e.g., decay, photosynthesis, algal respiration), boundary losses such as stream benthic deposits, and boundary gains (e.g., reaeration, and point or distributed waste discharges). Assuming that the longitudinal dispersion coefficient and the advective velocity are constant along the flow axis, equation (3.28) may be expanded for the generalized component in Figure III-1 to:

\[
\frac{3C}{3t} = E \frac{3^2C}{3x^2} - U \frac{3C}{3x} - KC + \frac{q_1}{A} (c_1 - C)
\]

(3.29)

where \( K \) = first-order reaction rate coefficient, 1/T,
\( q_1 \) = influent fluid flow rate per unit length, L/T,
\( c_1 \) = concentration of water quality parameter in the inflow, M/L^3,
and
\( A \) = wetted cross-sectional area in the component, L^2.

In equation (3.29) the influent fluid flow rate per unit length, influent concentration, and the wetted cross-sectional area may all be variable functions of time.

Solutions to differential equations derived from equation (3.28), governing the behavior of conventional dry-weather flow storage/treatment systems and receiving waters, are presented in detail elsewhere (Medina, 1982) as part of the development of the Storage/Treatment and Receiving Water Quality Frequency and Duration Model (STO/TRT RECEIVING). This model has been proposed in Chapter I (Table I-1) at the highest level of analysis in the hierarchical, integrated methodology for assessing instream water quality. The model LEVEL III - RECEIVING constitutes the more simplified steady-state version, without detailed simulation of point source storage and treatment.

Receiving Body of Water

A non-tidal receiving stream can be adequately represented by a one-dimensional, advective-dispersive system with a constant, uniform cross-sectional area, constant velocity and longitudinal dispersion--over each stream segment (Δx) and time step (Δt). The governing equations for the biochemical oxygen demand (BOD)-dissolved oxygen (DO) coupled reaction are derived from equation (3.28) for continuously-discharging plane sources in the y-z plane:
where $L = \text{BOD concentration, M/L}^3$

$K_{13} = \text{biochemical oxidation rate and sedimentation rate coefficient for carbonaceous BOD, 1/T,}$

$D = \text{DO deficit } = C - C_s, \text{ M/L}^3,$

$C_s = \text{saturation concentration of DO at stream temperature, M/L}^3,$

$C = \text{concentration of DO in stream, M/L}^3,$ and

$K_2 = \text{reaeration rate coefficient, 1/T.}$

The solutions to equations (3.30) and (3.31) have been derived elsewhere (Medina, 1982). The BOD concentration is given by:

$$L = \frac{xU}{2\pi^2} \cdot \frac{e^{-x_t}}{2\pi x} \left[ \left\{ \text{erf} \left( \frac{x + \Omega t}{\sqrt{4E_x t}} \right) - \text{erf} \left[ \frac{x + \Omega(t - t_1)}{\sqrt{4E_x(t - t_1)}} \right] \right\} \exp \left( \frac{x\Omega}{2E_x} \right) ight.$$  

$$\left. \frac{xU}{2\pi^2} \cdot \frac{e^{-x_t}}{2\pi x} \left[ \left\{ \text{erf} \left( \frac{x - \Omega t}{\sqrt{4E_x t}} \right) - \text{erf} \left[ \frac{x - \Omega(t - t_1)}{\sqrt{4E_x(t - t_1)}} \right] \right\} \exp \left( -\frac{x\Omega}{2E_x} \right) \right] \right]$$  

\[3.32\]
where \( q'' \) = time rate of pollutant mass injection per unit area at the plane source, \( M/L^2 T \),
\( c_1 \) = concentration of pollutant in the inflow, dimensionless,
\( E_x \) = longitudinal dispersion coefficient, \( L^2 / T \),
\( t_1 \) = period of injection, \( T \),
\( \Omega = \sqrt{U^2 + 4K_{13}E_x} \), with dimensions of velocity, \( L/T \), and
\( \rho \) = density of the receiving fluid, \( M/L^3 \).

The continuous plane source solution for DO deficit is given by (Medina, 1982):

\[
D = \left[ p'' - \frac{K_{13}q''}{K_2-K_{13}} \right] \left[ \frac{xU}{2E_x} \right] \left[ \left\{ \text{erf} \left( \frac{x+\Omega t}{\sqrt{4E_x t}} \right) - \text{erf} \left( \frac{x+\Omega(t-t_1)}{\sqrt{4E_x(t-t_1)}} \right) \right\} \exp \left( \frac{x\Omega}{2E_x} \right) \right.
\]

\[
- \left\{ \text{erf} \left( \frac{x-\Omega t}{\sqrt{4E_x t}} \right) - \text{erf} \left( \frac{x-\Omega(t-t_1)}{\sqrt{4E_x(t-t_1)}} \right) \right\} \exp \left( \frac{-x\Omega}{2E_x} \right) \right]
\]

\[
+ \frac{xU}{2E_x} \left[ \left\{ \text{erf} \left( \frac{x+\Omega t}{\sqrt{4E_x t}} \right) - \text{erf} \left( \frac{x+\Omega(t-t_1)}{\sqrt{4E_x(t-t_1)}} \right) \right\} \exp \left( \frac{x\Omega}{2E_x} \right) \right.
\]

\[
- \left\{ \text{erf} \left( \frac{x-\Omega t}{\sqrt{4E_x t}} \right) - \text{erf} \left( \frac{x-\Omega(t-t_1)}{\sqrt{4E_x(t-t_1)}} \right) \right\} \exp \left( \frac{-x\Omega}{2E_x} \right) \right]
\]

\[
(3.33)
\]
where \( p'' \) = time rate of mass injection of DO deficit per unit area at the plane source, \( \frac{M}{L^2 T} \), and

\[
\Omega_2 = \sqrt{U^2 + 4K_2 E_x}, \quad \text{with dimensions of velocity,} \ L/T.
\]

Equations (3.32) and (3.33) are used in STO/TRT RECEIVING to predict BOD and DO concentrations in space and time.

For steady-state analysis, as implemented in LEVEL III-RECEIVING, equations (3.30) and (3.31) reduce to:

\[
U \frac{\partial L}{\partial x} = E_x \frac{\partial^2 L}{\partial x^2} - K_{13} L \tag{3.34}
\]

\[
U \frac{\partial D}{\partial x} = E_x \frac{\partial^2 D}{\partial x^2} + K_{13} L - K_2 D \tag{3.35}
\]

The solution to equation (3.34) is

\[
L = L_0 \exp \left[ \frac{xU}{2E_x} \left( 1 - \sqrt{1 + \frac{4K_{13} E_x}{U^2}} \right) \right] \tag{3.36}
\]

where \( L_0 \) = ultimate first-stage BOD demand, mg/l.

for \( x \geq 0 \). The solution to the governing differential equation for steady-state dissolved oxygen deficit becomes:

\[
D = \frac{L_0 K_{13}}{K_2 - K_{13}} \left[ e^{j t} - \frac{s_1}{s_2} e^{s_2 t} \right] + D_0 e^{s_2 t} \tag{3.37}
\]

where \( t = \frac{x}{U} \) = lapsed time, hours,

\[
j = \frac{U^2}{2E} \left[ 1 - \sqrt{1 + \frac{4K_{13} E}{U^2}} \right], \text{ hours}^{-1},
\]

38
\[ g = \frac{U^2}{2E} \left[ 1 - \sqrt{1 + \frac{4K_pE}{U^2}} \right], \text{ hours}^{-1}, \]

\[ s_1 = \sqrt{1 + \frac{4K_{13}E}{U^2}}, \text{ dimensionless}, \]

\[ s_2 = \sqrt{1 + \frac{4K_pE}{U^2}}, \text{ dimensionless}, \text{ and} \]

\[ D_0 = \text{initial DO deficit in the stream, mg/L}. \]

### 3.4 Key Deterministic Water Quality Model Parameters

Several key parameters in STO/TRT RECEIVING and LEVEL III-RECEIVING must be either be supplied by the user as input data or calculated by the program as a function of other model variables. Some of these parameters may be used in the calibration procedure with field-measured data.

**Stream Depth and Velocity**

Depth and velocity of flow are key hydraulic parameters, particularly with respect to instream flow studies (Bovee and Milhous, 1978). The classical work of Leopold and Maddock (1953) demonstrated that an approximation can be made which uses strong correlations between velocity versus flow and depth versus flow, for example:

\[ H = a_1 Q^{a_2} \quad (3.38) \]

where \( Q = \text{streamflow, cfs}, \) and \( a_1, a_2 = \text{regression coefficients}. \)

Equation (3.38) is a power function, and represents a stage-discharge relationship. The user may indeed choose this equation to determine stream depth from known discharge (from the hydrologic simulation model). Two other curve fitting options may be used:
(1) parabolic regression

\[ H = \alpha_1 + \alpha_2 Q + \alpha_3 Q^2 \]  
(3.39)

(2) a combination of power curve fitting and linear regression

\[ H = \alpha_1 Q^{\alpha_2}, \quad Q < Q^* \]  
(3.40)

\[ H = \beta_1 + \beta_2 Q, \quad Q \geq Q^* \]  
(3.41)

where \( Q^* \) is the point at which the two curves join and \( \alpha_3, \beta_1, \) and \( \beta_2 \) are the other regression coefficients. Any of the above regression coefficients are easy to determine from a given stage-discharge relationship, including the degree of correlation of the fit, and programs for hand-held calculators are readily available.

Nevertheless, a fourth and superior option is to supply the models with the values of the rating table, and to compute stream depth from discharge by divided-difference interpolation using the Newton form for the interpolating polynomial, the reverse process of that discussed previously in Section 3.2, equation (3.24).

Stream velocity is subsequently computed, at every hourly time step, as a function of depth, slope of the river profile and channel roughness using the Manning equation:

\[ V = \frac{1.49}{n} H^{2/3} S^{1/2} \]  
(3.42)

where \( H \) = stream depth, approximately equal to hydraulic mean radius for natural river sections, ft

\( S \) = bed slope, dimensionless

\( n \) = Manning's river channel roughness, dimensionless, and

\( V \) = mean velocity, ft/sec.

It is recognized that the slope \( S \) in equation (3.42) should be the slope of the energy grade line. For practical purposes, and short time steps (1 hour), the equation is sufficiently accurate.
Longitudinal Dispersion Coefficient

The longitudinal dispersion coefficient characterizes the combined effects of both dispersion and diffusion. It can either be supplied as a constant, based on data or other available information, or it can be calculated. If calculated, the computations are performed for each hourly time step according to (Liu, 1977):

\[ D_L = \beta \frac{Q^2}{U_s R^3} \tag{3.43} \]

where \( D_L \) = longitudinal dispersion coefficient,
\( R \) = hydraulic radius of stream,
\( U_s \) = frictional or shear velocity
\[ = \sqrt{g RS}, \]
\( g \) = gravitational constant, and
\( S \) = slope of stream, and
\( \beta = 0.18 \left( \frac{U_s}{V} \right)^{1.5} \tag{3.44} \)

where \( V \) = mean flow velocity.

Inherent in this approach is the assumption that
\[ \frac{A}{W} = H_m = R \tag{3.45} \]

for a wide channel, where
\( A \) = area of channel cross-section
\( W \) = width of channel, and
\( H_m \) = mean depth of flow.

This allows substitution of \( R \) for \( H_m \). Conversely, the formula to predict dispersion becomes:

\[ E_x = \beta \frac{Q^2}{U_s H^3}, \tag{3.46} \]
where \( E_x \) = longitudinal dispersion coefficient, \( ft^2/sec \), 
\( Q \) = total flow, cfs, 
\( H \) = mean depth of flow, ft, 
\( U_* \) = shear velocity, \( ft/sec \) 
\[ \sqrt{g HS} \] 
where \( g \) = gravitational constant, 
\[ = 32.174 \, ft/sec^2 \] 
and 
\( S \) = stream slope, dimensionless, and 
\( V \) = mean flow velocity, \( ft/sec \).

**Deoxygenation and Reoxygenation Rates**

The deoxygenation coefficient, \( K_1 \), represents the loss of DO in the waterway due to reduction of BOD. It is expressed as a constant fraction of the remaining unoxidized organic matter in any arbitrary unit of time. The average domestic sewage deoxygenates at about 0.23 per day at 20°C under standardized laboratory conditions. In freshwater streams, the reaction coefficient for BOD ranges from 0.2 to 2.0 per day for water temperatures from 20°C to 25°C (Hydroscience, Inc., 1971). There are at least four generally accepted methods to determine the value of \( K_1 \) from the BOD curve, for a wastewater sample. These include: (1) the least-squares technique, (2) the slope method, (3) the moments method, and (4) the logarithmic method (Nemerow, 1974).

The magnitude of \( K_1 \) in streams is related to the average water depth. The explanation behind this correlation lies in the fact that the smaller the depth the greater the contact with biological film in the stream bed, one of the most important factors in natural oxidative processes (Hydroscience, Inc., 1971). From data reported in the literature, a straight-line plot between the variables is obtained (within certain bounds) as shown in Figure III-2. A mathematical representation is given by:

\[ K_1 = \gamma_1 H^2 \]  

(3.47)

where \( K_1 \) = deoxygenation coefficient at 20°C, day\(^{-1}\), 
\( \gamma_1 \), \( \gamma_2 \) = regression coefficients.

The above relationship appears reasonable within a range of depths from 1 foot to 10 feet. Thus, \( K_1 \) must be limited by program variables \( (XK1MAX \text{ and } XK1MIN) \) to upper and lower bounds, respectively. These may be selected by the user, and supplied to both models as input data, so as to further extend or restrict the range of applicability of equation (3.47) to suit local stream conditions. A temperature correction yields:

\[ K_1(T) = K_1(20^\circ) 1.047^{T-20} \]  

(3.48)
Figure III-2. Deoxygenation Rate As A Function of Stream Depth (modified after Hydroscience, Inc., 1971). Equation shown is a specific function which corresponds to $\gamma_1 = 0.99$ and $\gamma_2 = -0.28$
where $T$ = water temperature, °C, and conversion is made to units of hour$^{-1}$ in the models. The magnitude of $X_{k1\text{MAX}}, X_{k1\text{MIN}}, \gamma_1$ and $\gamma_2$ may be adjusted during calibration procedures.

The reaeration rate, $K_r$, is also calculated during each hourly time-step. In STO/TRT RECEIVING, one of four formulas may be selected, or as another option, the model itself selects the best relationship on the basis of stream depth and velocity criteria (Covar, 1976). In LEVEL III-RECEIVING, only the Langbein-Durum formula below is implemented for simplicity. The four predictive relationships in STO/TRT RECEIVING are (Medina, 1982):

(1) the Owens-Edwards-Gibbs formula (1964)-

$$K_2 = 2.303 \frac{9.41 V^{0.67}}{H^{1.85}} \cdot \frac{1}{24}$$

(3.49)

(2) the Langbein-Durum formula (1967)-

$$K_2 = 2.303 \frac{3.3 V}{H^{1.33}} \cdot \frac{1}{24}$$

(3.50)

(3) the O'Connor-Dobbins formula (1958)-

$$K_2 = \left( \frac{D_m \cdot V \cdot 3600}{H^{1.5}} \right)^{0.5}$$

(3.51)

and

(4) the Churchill-Elmore-Buckingham formula (1962)-

$$K_2 = 2.303 \frac{5.026 V^{0.969}}{H^{1.673}} \cdot \frac{1}{24}$$

(3.52)

where, in all of the above,
The molecular diffusion coefficient, $D_m$, is equal to 0.000081 ft$^2$/hr (Thomann, 1972). The reaeration rate is adjusted for varying temperature according to

$$K_2(T) = K_2(20^\circ C) \cdot 1.024(T-20)$$  \hspace{1cm} (3.53)

where $T$ is the stream temperature during each hourly time step.

If the user chooses to let STO/TRT RECEIVING select the most appropriate formula for computing $K_2$, criteria set by (Covar, 1976) with regard to (1), (3) and (4) above are implemented. For all time steps during which the stream depth is less than two feet, the Owens-Edwards-Gibbs formula is used. At higher stream depths, the stream velocity determines whether the O'Connor-Dobbins or the Churchill-Elmore-Buckingham formula is selected. The "dividing line" between the two formulas can be approximated by the relationship

$$\text{depth} = 0.59 \cdot \text{velocity}^{2.63}$$ \hspace{1cm} (3.54)

such that if stream depth is less than or equal to that predicted by equation (3.88), the Churchill-Elmore-Buckingham formula is chosen. Otherwise, the O'Connor-Dobbins formula is used to predict $K_2$.

**Saturation Concentration of Dissolved Oxygen**

The saturation concentration is determined for both models from the regression relationship (Elmore and Hayes, 1960):

$$C_s = 14.652 - 0.41022(T) + 0.0079910(T)^2 - 0.000077774(T)^3$$ \hspace{1cm} (3.55)

where $T$ = water temperature, °C, and $C_s$ is in mg/L of dissolved oxygen. If a correction for barometric pressure and salinity are required, program modification is needed; however, equation (3.55) is adequate for most practical purposes, for standard pressure and zero salinity.
3.5 STATISTICAL WATER QUALITY MODELS

In essence, the statistical models calculate the probability distribution of receiving water pollutant concentrations, given the probability distribution of the model inputs (e.g., flows, pollutant loadings, etc.). Whereas the deterministic models attempt to predict the time histories of the output variables and perform frequency analyses on the basis of the historical series, probabilistic water quality analysis attempts to calculate the frequency distributions without computation of the exact sequence of events. Instead, the probability distributions and correlation structure of the input variables are used to compute directly the frequency distribution of the output variables. These statistical models are conceptual simplifications, which require only the statistical properties of the input time series (e.g., medians, means, coefficients of variations, cross-correlations).

The simplest mass balance that can be used to calculate river quality downstream from a pollutant discharge is:

\[
C_T = \frac{Q_s C_s + Q_p C_p}{Q_s + Q_p}
\]

(3.56)

where

- \( C_T \) = mixed pollutant concentration in the river, downstream from the pollutant discharge location
- \( Q_s \) = upstream river flow
- \( C_s \) = upstream pollutant concentration
- \( Q_p \) = point pollutant source flow rate, and
- \( C_p \) = point pollutant source concentration.

It is tempting to assume that appropriate statistics of upstream flow and quality can be substituted into equation (3.56) to produce specific mean or percentile values of \( C_T \) (Warn and Brew, 1980): for example, to use annual mean values of the upstream variables hoping this would give the annual mean downstream river quality. This kind of procedure is wrong in principle and will produce incorrect results for almost any real stream pollution problem (Warn and Brew, 1980). These investigators proposed an analytical method based on the first two moments (mean, variance) of a two-parameter log-normal distribution, assumed to represent adequately certain ratios of the variables in equation (3.56) above. For nonpoint source contributions, \( Q_R \) and \( C_R \) may replace \( Q_p \) and \( C_p \), and by regrouping the variables equation (3.56) becomes (DiToro), 1982):
The probability model that follows assumes that $Q_s$, $Q_R$, $C_s$, and $C_R$ are jointly lognormally distributed.

**Moments Approximation**

As noted by Warn and Brew (1980) and modified by DiToro (1982), the mass balance equation can be rewritten in the form:

$$C_T = C_R \phi + C_s (1 - \phi)$$

(3.58)

where

$$\phi = \frac{Q_R}{Q_R + Q_s} = \text{runoff flow fraction.}$$

If $\phi$ and $(1 - \phi)$ were lognormally distributed, and since $C_R$ and $C_s$ are also assumed to be lognormal, the products $C_R \phi$ and $C_s (1 - \phi)$ would also be lognormal. This is an approximation based on the fact that sums of lognormal random variables have been reported to have tails which are also approximately lognormal (DiToro, 1982 references Janos, 1970). These assumptions are shown to be quite good from data collected in nationwide studies (see section 4.7) and for the North Carolina study site (see section 5.2).

At any rate, these observations suggest that $C_T$ should be approximately lognormal, and the mean and variance suffice to determine its distribution. The arithmetic mean and variance of $C_T$ are given by (assuming independence):

$$\mu(C_T) = \mu(C_R) \mu(\phi) + \mu(C_s) [1 - \mu(\phi)]$$

and

$$\sigma^2(C_T) = \sigma^2(\phi) [\mu(C_R) - \mu(C_s)]^2$$

$$+ \sigma^2(C_R) [\sigma^2(\phi) + \mu^2(\phi)]$$

$$+ \sigma^2(C_s) [\sigma^2(\phi) + (1 - \mu(\phi))^2]$$

(3.59)

(3.60)

where $\mu(\ ) = \text{mean of ( )}$

$\sigma^2(\ ) = \text{variance of ( )}$
since it is assumed that both concentrations, \( C_S \) and \( C_R \), are uncorrelated with respect to their respective flows, \( Q_S \) and \( Q_R \), and therefore also the runoff fraction \( \phi \). The exact distribution of \( C_T \) is discussed in the next section, but it should be pointed out that both statistical models predicted an almost identical stream BOD concentration frequency distribution for the North Carolina study site (see Chapter V).

If \( C_T \) is assumed lognormal, then the relationships between the arithmetic moments, \( \mu(C_T) \) and \( \sigma^2(C_T) \), and the log mean, \( \mu_\ell \), and log standard deviation, \( \sigma_\ell \), are:

\[
\mu_\ell(C_T) = \ln \left[ \frac{\mu(C_T)}{\sqrt{1 + \nu^2(C_T)}} \right] 
\]

and

\[
\sigma_\ell^2(C_T) = \ln \left[ 1 + \nu^2(C_T) \right] 
\]

where

\[
\nu(C_T) = \frac{\sigma(C_T)}{\mu(C_T)} 
\]

= coefficient of variation of \( C_T \).

The quantiles of \( C_T \) are:

\[
C_{T\alpha} = \exp \left[ \mu_\ell(C_T) + z_\alpha \sigma_\ell(C_T) \right] 
\]

which is the concentration that is exceeded with probability \( (1 - \alpha) \), where \( z_\alpha \) is the standard normal \( \alpha \) quantile.

The remaining task is to compute the moments of \( \phi \). Warn and Brew (1980) suggest a numerical integration. However, DiToro (1982) notes that, since the method is approximate, numerical techniques are best reserved for the evaluation of the exact distribution of \( C_T \). He derived the following approximate expressions:

\[
\mu_\ell(\phi) = \frac{1}{2} \left[ \ln(\phi_\alpha) + \ln(\phi_{1 - \alpha}) \right] 
\]

and

(3.65)
He chose \( z_\alpha = 1.645 \) to force agreement of this straight-line approximation of the 5% and 95% quantiles. Once the log mean and standard deviation are computed from equations (3.65) and (3.66), then the arithmetic moments follow from the lognormal assumption:

\[
\mu(\phi) = \exp \left[ \mu_\zeta(\phi) + \frac{1}{2} \sigma_\zeta^2(\phi) \right] \tag{3.67}
\]

and

\[
\nu^2(\phi) = \exp \left[ \sigma_\zeta^2(\phi) \right] - 1 \tag{3.68}
\]

where

\[
\sigma(\phi) = \mu(\phi) \nu(\phi)
\]

These arithmetic moments of \( \phi \) are subsequently used in equations (3.59) and (3.60) to compute the arithmetic moments of \( C_T \). A computer program written in BASIC, modified from Driscoll (1981), is presented in Appendix G.

**Gaussian Quadrature**

The probability of \( C_T \) exceeding a value \( C_T^* \) can be expressed as a multiple integral of the joint probability density over the values of flows and concentration for which \( C_T \) is greater than \( C_T^* \). This requires an integral for each of the variables \( C_R \), \( Q_R \), \( C_s \) and \( Q_s \) (DiToro, 1982). One integral can be eliminated by combining \( Q_s \) and \( Q_R \) into the ratio \( D = Q_s/Q_R \) so that:

\[
\text{Prob} \{ C_T > C_T^* \} = \text{Prob} \left\{ \frac{C_R}{1 + D} + \frac{C_s D}{1 + D} > C_T^* \right\} \tag{3.69}
\]

Since \( Q_s \) and \( Q_R \) are both lognormal, so is the ratio \( D \). For a fixed \( C_T^* \), the equality

\[
C_T^* = \frac{C_R}{1 + D} + \frac{C_s D}{1 + D} \tag{3.70}
\]

defines a surface in \( C_R \), \( C_s \) and \( D \) space and the required probability is the integral of the joint probability density function above
this surface. Considering a fixed ratio $D$, then

$$C_R + C_s D = C_T^*(1 + D) \quad (3.71)$$

The limits of the integrals for the exceedence probability are:

$$\text{Prob} \{ C_T > C_T^* \} = \int_{C_R}^{C_T^*} \left[ \int_{C_S}^{C_T^*} \int_{D=0}^{C_T^*-C_R} f(C_R, C_S, D) \, dC_R \, dC_S \right] dC_T$$

$$+ \int_{C_T^*}^{C_T} \int_{C_S}^{C_T^*} f(C_R, C_S, D) \, dC_R \, dC_S \, dD \quad (3.72)$$

where

$$C_{R1} = C_T^* (1 + D) - C_D$$

$$C_{S1} = C_T^* (1 + 1/D)$$

and $f(C_R, C_S, D)$ is the joint probability density function for $C_R, C_S$ and $D$. A more convenient expression can be obtained in terms of the natural logarithms of the variables so that the joint probability density function $f(C_R, C_S, D)$ is trivariate gaussian. A computer program, based on gaussian quadrature, is used to obtain the frequency distribution of $C_T$ and is presented in Appendix J.
CHAPTER IV

DESCRIPTION OF STUDY AREA
AND HYDROLOGIC TIME SERIES

Even a thorough description of the physical characteristics of a drainage basin is of limited value for water quantity, water quality, and floodplain management and planning -- without a conjunctive review of climatology and historical hydrologic inputs. Coupling such inputs with information about topography, drainage, land use and demographic projections results in a better understanding of the stresses and demands placed upon a valuable natural resource which has been generally available in sufficient quantity and quality to support a wide variety of uses: domestic water supply, industry, agriculture, recreation, waste transport and assimilation.

4.1 YADKIN-PEE DEE RIVER BASIN

The Yadkin-Pee Dee River basin extends from Virginia, through central North Carolina, into South Carolina as illustrated by the shaded area in Figure IV-1. Most of that portion of the basin which lies in North Carolina is in the upper and middle Piedmont physiographic region. The Piedmont lies between the Coastal Plain and the Appalachian Mountains, and includes about two-fifths of the land area of North Carolina. The topography consists of rounded hills and long, rounded ridges with a northeast-southwest trend. The Yadkin River originates on the generally steep slopes of the Blue Ridge Mountains of North Carolina, with elevations exceeding 3500 feet (1067 meters). The river flows east for about 100 miles (161 km) before turning sharply south near Winston-Salem, North Carolina, as shown in Figure IV-2. In south-central North Carolina, the Yadkin River joins the Uwharrie River from the east, upstream of Lake Tillery. Downstream from their confluence, the river is known as the Pee Dee River. Most of the rest of the basin is in the Coastal Plain, including major tributaries such as the Lumber River, which drains southeastern North Carolina and joins the Pee Dee River in eastern South Carolina. The combined drainage area of the Yadkin-Pee Dee and Lumber Rivers is 10,556 mi² (27,340 km²).

Streams within the Piedmont urban areas are relatively small with well entrenched channels and sandy bottoms. In a few locations, stream channels are cut into bedrock. Streams are also fairly steep with main channel slopes of more than 15 feet per mile and small tributary slopes of over 100 feet per mile (Putnam, 1972). In the natural state, most flood plains are covered with a dense growth of brush. However, encroachment of extensive developments has caused drainage and flooding problems, which are expected to increase. Of particular interest in this study are both Salem Creek and Muddy Creek. Salem Creek drains
Figure IV-1. Yadkin-Pee Dee River Basin, North Carolina, South Carolina and Virginia

Figure IV-2. The Main Branch of the Yadkin-Pee Dee River in North Carolina
much of the city of Winston-Salem and is a tributary to Muddy Creek which discharges into the Yadkin River (sketched in Figure IV-2).

Wastes entering the Yadkin River from the Winston-Salem area, particularly during heavy rains, resulted in several major fish kills in the late 1960's and the early 1970's (Lindskov, 1974). A more recent incident, not connected to a storm event, is described in a later section.

4.2 CLIMATE

North Carolina lies between 33° and 37° north latitude, and between 75° and 84° west longitude. The span of longitude is greater than that of any other state east of the Mississippi River, with its greatest length (from east to west) being 503 miles (805 km). It also has the greatest range of altitude and the most varied climate of any eastern state (Hardy, 1970). This is due primarily to its wide range in elevation and distance from the ocean, with lesser influences due to: latitude, inland bodies of water, soil surface and plant cover. In all seasons of the year the average temperature varies more than 20° from the warmer lower coast to the colder highest mountain elevations.

Altitude has an important effect on rainfall. Orographic precipitation is associated with a cooling process due to lifting of moist horizontal air masses over natural topographic barriers such as mountain ranges (e.g., Pacific Northwest). The rainiest part of the eastern United States, with an annual average of more than 80 inches (2032 mm), is located in southwestern North Carolina where moist southerly winds are forced upward in passing over the Great Smokies and the Blue Ridge Range. East of the mountains, average annual rainfall ranges mostly from 40 to 55 inches (1016 to 1397 mm). The annual average precipitation at the Greensboro-Winston-Salem National Oceanic and Atmospheric Administration (NOAA) first-order weather station, based on a period of record from 1931 to 1960, is 42.16 inches (1071 mm). It is the hourly rainfall station, with a long period of record, which is closest to the study site. An annual average temperature of 58.2°F (14.56°C) was reported for the same period of record (Hardy, 1970). Hardy (1970) offers an excellent description of seasonal precipitation:

"There are no distinct wet and dry seasons in North Carolina. There is some seasonal variation in average precipitation. Summer rainfall is normally the greatest, and July the wettest month. Since the rain at this time of year comes mostly with thunderstorms and convective showers, it is also more variable than at other seasons. Daily showers are not uncommon, nor are periods of one or two weeks without rain. Autumn is the driest season, and October the driest month. Precipitation in winter and spring occurs mostly with
migratory low pressure storms. It appears with greater regularity and more even distribution than summer showers.

Winter precipitation usually occurs with southeasterly through easterly winds, and is seldom associated with very cold weather. Snow and sleet occur on an average of once or twice a year near the coast, and not much more often over the southeastern half of the State. Such occurrences are nearly always connected with northeasterly winds, generated when high pressure over the interior of the northeastern United States causes a flow of cold air down parallel to the coastline, while offshore low pressure brings in cool, moist air from the North Atlantic. Over the Mountains and western Piedmont frozen precipitation sometimes occurs with interior low pressure storms. In the extreme west it can happen with a cold front passage from the northwest. Average winter snowfall ranges from about 1 inch per year on the Outer Banks and the lower coast, to about 9 inches in the northern Piedmont and southern Mountains. Some of the higher mountain peaks and upper slopes receive an average of nearly 50 inches a year."

and also some general comments on flooding and summer thunderstorms:

" Floods occur frequently, affecting some part of North Carolina each year. Loss of life is rare, and the economic loss not generally large, but the cost of floods is increasing as river lowgrounds are developed. Floods may occur at any season, but are most frequent in early spring, summer and early fall. Rains associated with West Indian hurricanes are the main cause of summer and fall floods. In mid-August 1940, severe floods occurred as a result of hurricane rains. Later in the same month intense rains of local origin caused severe flooding in western North Carolina. Major floods also occurred in September 1945, October 1929, August 1928, and July 1916.

The greatest economic loss entailed in North Carolina because of stormy weather is that due to summer thunderstorms. These usually affect only limited areas, but hail and wind occurring with some of them account for an average yearly loss of about $5 million. Three to five people are killed in the State by lightning during the average year. Farm livestock, especially cattle are killed in larger numbers, and there is a considerable loss of property due to fires set by lightning. In any given locality, 40 to 50 days with thunderstorms may
be expected in a year."

In a comprehensive study of the effect of urban development on floods in the Piedmont Province of North Carolina (Putnam, 1972), it was concluded that: the peak discharge can be expected to increase by a factor of about two to four, depending upon the recurrence interval of the flood and the anticipated conditions of development. Because North Carolina was largely an agricultural state until well after the turn of the century, cities and towns in the interior developed on the uplands between streams. Thus, even major floods such as the two that affected 20,000 square miles (51,800 square kilometers) in August 1940, caused less than $10 million in damages and 26 deaths (Heath, 1978). Increased encroachment into the floodplains, due to an increasing population and a more highly industrialized economic base, has created flood-prone areas faster than protection can be provided. Consequently, the flood of November 1977 seriously affected 1800 square miles (4662 km²) in western North Carolina and resulted in the loss of 11 lives, 384 homes, 12 dams on small ponds, 389 miles (622 km) of highway, and 100 bridges. Total property damage exceeded $45 million (Heath, 1978). Droughts, the other extreme hydrologic events, deplete water supplies for domestic, industrial and agricultural uses and often impair recreation by lowering lake levels and exposing bottom debris. They also affect the waste assimilative capacity of streams by reducing dilution. In 1977, not an unusually dry year in North Carolina, 60 counties were declared agricultural-disaster areas. According to Heath (1978), the first drought-related problems in North Carolina will arise in the Piedmont urban areas that are located in the headwaters of major river basins. Additional water supplies can be developed only downstream, also the direction in which wastes move.

4.3 WATER QUALITY OF YADKIN-PEE DEE RIVER, NORTH CAROLINA

According to Harned and Meyer (1981), overall ambient water quality in the Yadkin-Pee Dee River is satisfactory for most water uses. Their conclusions are based upon statistical analysis of data collected by the U. S. Geological Survey at three stations: Yadkin River at Yadkin College (02116500), Rocky River near Norwood (02126000), and Pee Dee River near Rockingham (02129000) -- which were sampled over different periods of time beginning in 1906. An expanded program of water-quality data collection, however, did not begin until 1973 at the three stations. Prior to 1973, samples were analyzed only for major ions, dissolved solids, hardness, specific conductance and pH. A continuous-recording monitor was used to measure dissolved oxygen at Yadkin College from 1971 to 1976, as well as temperature, specific conductance and pH. Harned and Meyer (1981) reported that iron and manganese are often above desirable levels; lead concentrations periodically rise above recommended criterion for domestic water use; mercury concentrations frequently exceed, and pH levels fall below, recommended criteria for protection of aquatic life;
dissolved oxygen levels, while generally good, are lowest near Rocking-
ham (near the North Carolina-South Carolina state line). Suspended sedi-
ment appears to be the most significant water quality problem, but a
dramatically decreasing trend in suspended sediment concentration has
been observed at Yadkin College since 1951 (daily sediment samples have
been collected at Yadkin College since 1950). It has been attributed to
changes in agricultural practices and land use in the basin.

A double-peaked response of suspended sediment concentration to
stormflows was observed to be characteristic at the Yadkin College station.
The first peak is caused by flushing of sediments from Muddy Creek, and
the second peak is the response of the Yadkin River itself. The con-
centration from the Muddy River was found to occur before the peak in the
hydrograph: demonstrating the commonly observed first-flush effect in
stormwater quality studies in receiving waters downstream from urban
areas (in the case above, City of Winston-Salem).

As early as June 1975, the National Eutrophication Survey (initiated
in 1972) identified High Rock Lake (downstream from Salisbury, North Caro-
lina) as eutrophic due to high concentrations of nutrients, with phosphor-
us as the limiting nutrient (U. S. Environmental Protection Agency, 1975).
A study conducted by Weiss and Kuenzler (1976) evaluated High Rock Lake
and classified it as 8-Eutrophic: on the basis of expected quality of
recreational water usage, the lake was considered poor for body contact
water sports and excellent as to probable fishing potential. While fish-
ing potential increases as the trophic status worsens to hypereutrophic,
fish kills may occur because of low oxygen levels at night or following
prolonged periods of cloud cover. There is also a fish species shift to
those considered coarse.

A comprehensive water resources study (Level B) was authorized by
the U. S. Water Resources Council (1979) for the Yadkin-Pee Dee River
Basin in cooperation with the environmental management staffs of the
States of North Carolina and South Carolina. The study concluded that
although water quality is generally good in the Upper Piedmont segment
of the Yadkin River, a stretch of stream from Kernersville, North Caro-
lina to Yadkin College was moderately to severely degraded. The stretch
includes Salem Creek and Muddy Creek: fecal coliform bacteria and metals
were found to be a significant water quality problem, with scattered
degradation with respect to dissolved oxygen and amonia nitrogen. These
problem areas appear to be degrading further, with improvements or stable
conditions being the exception rather than the rule.

Water Uses and Waste Disposal

The North Carolina Department of Natural and Economic Resources
(now the N. C. Department of Natural Resources and Community Development)
conducted an inventory of all major point sources (municipal and
industrial) in the Yadkin-Pee Dee River Basin as part of a water quality management plan (N. C. DNER, 1976). Almost 70 percent of the total wastewater flow into the river system is contributed by 16 large sources, listed in Table IV-1. Computerized access to the Hydrologic Storage and Retrieval System (HISARS) yielded a 7-day, 10 year low flow (7Q10) of 15.51 ft³/sec (0.44 m³/sec) and an average flow of about 73 ft³/sec (2.07 m³/sec) at Salem Creek near Atwood, North Carolina. As noted in Table IV-1, the treatment plant design flow effluent is much larger than the 7Q10 and would constitute about 43 percent of the total flow in Salem Creek, downstream from the plant outfall, during average flow conditions. These streamflow values are based on the entire time series stored in HISARS from U.S.G.S. records at station no. 02115856: October 1971 to October 1981. By comparison, the 7Q10 at Muddy Creek near Muddy Creek, North Carolina (U.S.G.S. station no. 02115860) is 55.39 ft³/sec (1.57 m³/sec) and the average flow 230 ft³/sec (6.51 m³/sec) based on a period of record from July 1964 to September 1979 (after which flow records were discontinued). This station is 0.2 mi (0.3 km) downstream from Muddy Creek's confluence with Salem Creek. Flow is regulated at the W. Kerr Scott Dam (near headwaters of the Yadkin River) so that a minimum flow of 700 ft³/sec (19.82 m³/sec) is maintained at Yadkin College gage (see Figure IV-3), slightly higher than the average 7Q10 value of 656 ft³/sec (18.58 m³/sec) computed by HISARS, based on the period of record from August 1928 to October 1981. The average flow is about 3000 ft³/sec (85 m³/sec). The Yadkin College gage is 18.1 mi (29 km) downstream from the Archie Elledge Sewage Treatment Plant effluent outfall.

Stream Water Quality Classifications and Standards

Two elements are involved in defining the quality to be maintained in a receiving body of water, or segment thereof:

1. intended or desired use of that body of water, its classification; and
2. the levels of contaminants that can be tolerated without impairing use of that body of water according to its classification, the standards.

About 18 years ago, North Carolina had streams designated as Class E, for agricultural and industrial processing and for waste transport (Division of Environmental Management, 1979). The corresponding standards were only stringent enough to protect against health hazards. Other stream segments were designated Class D: standards were sufficient to allow fish to survive, but not to propagate. All waters of the State are now classified "C" or better: their minimum designated use is for fish propagation. However, classification does not insure that the pollutant levels are low enough to be meeting corresponding stream standards. State of North Carolina classifications are presented in Table IV-2. The corre-
TABLE IV-1.--Major Municipal and Industrial Wastewater Discharges To
the Yadkin-Pee Dee River (modified from North Carolina
Department of Natural and Economic Resources, 1976)

<table>
<thead>
<tr>
<th>Facility</th>
<th>Location</th>
<th>Location of waste discharge</th>
<th>Design flow ft³/s (m³/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Wilkesboro Municipal</td>
<td>Wilkes County</td>
<td>Cub Creek</td>
<td>5.1 (0.14)</td>
</tr>
<tr>
<td>2. Chatham Manufacturing Company, Elkin</td>
<td>Surry County</td>
<td>Yadkin River</td>
<td>6.2 (0.18)</td>
</tr>
<tr>
<td>3. Elkin Municipal</td>
<td>Surry County</td>
<td>Yadkin River</td>
<td>4.7 (0.13)</td>
</tr>
<tr>
<td>4. Mt. Airy Municipal</td>
<td>Surry County</td>
<td>Ararat River</td>
<td>24.8 (0.70)</td>
</tr>
<tr>
<td>5. Salisbury Municipal</td>
<td>Rowan County</td>
<td>Grants Creek</td>
<td>7.8 (0.22)</td>
</tr>
<tr>
<td>6. NC Finishing Company</td>
<td>Rowan County</td>
<td>High Rock Lake</td>
<td>6.6 (0.19)</td>
</tr>
<tr>
<td>7. Duke Power Company</td>
<td>Rowan County</td>
<td>High Rock Lake</td>
<td>7.0 (0.20)</td>
</tr>
<tr>
<td>8. Winston-Salem Municipal</td>
<td>Forsyth County</td>
<td>Salem Creek **</td>
<td>55.8 (1.58)</td>
</tr>
<tr>
<td>9. Statesville Municipal</td>
<td>Iredell County</td>
<td>Third Creek</td>
<td>6.2 (0.18)</td>
</tr>
<tr>
<td>10. High Point Municipal</td>
<td>Davidson County</td>
<td>Rich Fork Creek</td>
<td>6.2 (0.18)</td>
</tr>
<tr>
<td>11. Thomasville Municipal</td>
<td>Davidson County</td>
<td>Hamby Creek</td>
<td>6.2 (0.18)</td>
</tr>
<tr>
<td>12. Mooresville Industrial</td>
<td>Iredell County</td>
<td>Dye Branch</td>
<td>6.2 (0.18)</td>
</tr>
<tr>
<td>13. Cannon Mills Company</td>
<td>Kannapolis</td>
<td>Dye Branch</td>
<td>26.4 (0.75)</td>
</tr>
<tr>
<td>14. Concord Regional</td>
<td>Cabarrus County</td>
<td>Rocky River</td>
<td>37.2 (1.05)</td>
</tr>
<tr>
<td>15. Monroe Municipal</td>
<td>Monroe County</td>
<td>Richardson Creek</td>
<td>7.0 (0.20)</td>
</tr>
<tr>
<td>16. Rockingham Municipal</td>
<td>Richmond County</td>
<td>Hitchcock Creek</td>
<td>9.3 (0.26)</td>
</tr>
</tbody>
</table>

** Note that the Archie Elledge wastewater treatment plant effluent, the largest source, is 3.6 times the magnitude of the 7-day, 10-year low flow (7Q10) computed by HISARS (Wiser, 1975) and based on the period of record from May 1971 to October 1981 (using the Log-Pearson Type III probability distribution) at U.S. Geological Survey streamflow station no. 02115856, Salem Creek near Atwod, N.C., 2700 feet (820 m) upstream of outfall.

Total for basin: 325.3 (9.21)
Figure IV-3. Yadkin River. U.S. Geological Survey Streamflow and Water Quality Station, Near Yadkin College, at Intersection with U.S. Highway 64.
<table>
<thead>
<tr>
<th>Classification</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Fresh Waters</strong></td>
<td></td>
</tr>
<tr>
<td>Class A-I:</td>
<td>Suitable as a source of water supply for drinking, culinary, or food processing purposes after treatment by approved disinfection only, and any other usage requiring waters of lower quality;</td>
</tr>
<tr>
<td>Class A-II:</td>
<td>Suitable as a source of water supply for drinking, culinary, or food processing purposes after approved treatment equal to coagulation, sedimentation, filtration, and disinfection, etc., and any other usage requiring waters of lower quality;</td>
</tr>
<tr>
<td>Class B:</td>
<td>Suitable for outdoor bathing, boating and wading, and any other usage requiring waters of lower quality;</td>
</tr>
<tr>
<td>Class C:</td>
<td>Suitable for fish and wildlife propagation; also suitable for boating, wading, and other uses requiring waters of lower quality.</td>
</tr>
<tr>
<td><strong>Tidal Salt Waters</strong></td>
<td></td>
</tr>
<tr>
<td>Class SA:</td>
<td>Suitable for shellfishing for market purposes and any other usage requiring waters of lower quality;</td>
</tr>
<tr>
<td>Class SB:</td>
<td>Suitable for bathing and any other usage except shellfishing for market purposes;</td>
</tr>
<tr>
<td>Class SC:</td>
<td>Suitable for fishing, and any other usage except bathing and shellfishing for market purposes.</td>
</tr>
</tbody>
</table>
Corresponding stream standards are presented in Table IV-3 for fresh waters.

Oxygen is as essential to aquatic life as it is to human life. As shown in Table IV-3, a level of 5 mg/l of dissolved oxygen (DO) is required to sustain acceptable biology in a stream; however, higher levels of DO are needed by sensitive game fish and many of the aquatic insects which provide a food source for them. The Division of Environmental Management (DEM) of the North Carolina Department of Natural Resources and Community Development (DNRCDC) has designated a DO level of 6.0 mg/l for trout waters. In Piedmont rivers only the headwater streams (in the mountains) are trout streams. DEM has placed that segment of the Yadkin River below its confluence with the Ararat River to High Rock Lake (and including Salem Creek and Muddy Creek) on the Degraded Streams List (DEM, 1979). The same stretch has also been identified as a biologically degraded stream segment: defined to contain degraded populations of fish (predominantly rough fish), as a result of surveys conducted by the N. C. Wildlife Resources Commission.

Stream use classifications will, of course, vary considerably from reach to reach, with changing land use, tributary inflows, waste sources, etc. A sample of such classifications is presented in downstream order in Table IV-4 for the Yadkin River and some of its tributaries, as obtained from the U. S. Environmental Protection Agency computerized Storage and Retrieval (STORET) system.

Fish Species

Duke Power Company conducted fish surveys downstream of Yadkin College gage (U.S. Highway 64) to High Rock Lake to meet requirements for drafting an environmental impact statement, for the proposed Perkins Nuclear Station. A total of 40 fish species was collected, 25 species (comprising 7 families) from the mainstream of the Yadkin River, from October 1973 to October 1974 (U.S. Nuclear Regulatory Commission, 1975). Minnows (which include the common carp) dominated the catch both in numbers (31 percent) and biomass (68.8 percent). Sunfishes were second in abundance numerically (28.6 percent), followed by catfishes (20.1 percent) and shads (17.7 percent). The single most abundant species was the bluegill (21.6 percent, a sunfish), followed by the common carp (18.2 percent). White bass migrate up the Yadkin River out of High Rock Lake in the early spring to spawn (U.S. Nuclear Regulatory Commission, 1975; Van Horne, 1982). Other species that migrate up the Yadkin River to spawn include redhorse suckers, white perch, channel and white catfish, and gizzard and threadfin shad (Van Horne, 1982).

Flow regulation such as the W. Kerr Scott Reservoir on the Yadkin has apparently limited smallmouth bass populations downstream. Large industrial waste discharges (see Table IV-1) as well as municipal discharges (Winston-Salem) continue to cause biological degradation.
TABLE IV-3. State of North Carolina Freshwater Stream Standards
(Division of Environmental Management, 1979; U.S.
Water Resources Council, 1979)

<table>
<thead>
<tr>
<th>Stream Use Classification</th>
<th>Fecal Coliform</th>
<th>Dissolved Oxygen</th>
<th>pH</th>
<th>Temperature</th>
</tr>
</thead>
<tbody>
<tr>
<td>A-I</td>
<td>50/100 ml</td>
<td>5.0 mg/l</td>
<td>Natural</td>
<td>Natural conditions.</td>
</tr>
<tr>
<td></td>
<td>(See Note 2)</td>
<td></td>
<td>conditions.</td>
<td></td>
</tr>
<tr>
<td>A-II</td>
<td>1000/100 ml</td>
<td>5.0 mg/l</td>
<td>6.0 - 8.5</td>
<td>Not to exceed 2.8 degrees above natural or exceed 32.2°C for Piedmont streams.</td>
</tr>
<tr>
<td></td>
<td>(See Note 3)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>B</td>
<td>200/100 ml</td>
<td>5.0 mg/l</td>
<td>6.0 - 8.5</td>
<td></td>
</tr>
<tr>
<td></td>
<td>(See Note 3)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>C</td>
<td>1000/100 ml</td>
<td>5.0 mg/l</td>
<td>6.0 - 8.5</td>
<td></td>
</tr>
<tr>
<td></td>
<td>(See Note 3)</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

1. Toxics standards not included.

2. Monthly average most probable number (MPN).

3. Logarithmic mean based on five consecutive samples examined during any one month.

<table>
<thead>
<tr>
<th>CLASS</th>
<th>STATION AND REACH IDENTIFICATION</th>
<th>COUNTY</th>
</tr>
</thead>
<tbody>
<tr>
<td>C</td>
<td>Yadkin River below Wilkesboro, N.C. (downstream from W. Kerr Dam)</td>
<td>Wilkes</td>
</tr>
<tr>
<td>C</td>
<td>Yadkin River at Elkin, N.C.</td>
<td>Surry</td>
</tr>
<tr>
<td>C</td>
<td>Ararat River near Siloam, N.C.</td>
<td>Surry</td>
</tr>
<tr>
<td>A-II</td>
<td>Salem Creek near Kernersville, N.C. (Salem Lake)</td>
<td>Forsyth</td>
</tr>
<tr>
<td>C</td>
<td>Salem Creek near Atwood, N.C. (USGS 02115856)</td>
<td>Forsyth</td>
</tr>
<tr>
<td>C</td>
<td>Muddy Creek near Clemmons Station, N.C. (USGS 02115860)</td>
<td>Forsyth</td>
</tr>
<tr>
<td>A-II</td>
<td>Yadkin River near Enon, N.C.</td>
<td>Forsyth</td>
</tr>
<tr>
<td>A-II</td>
<td>Yadkin River at Yadkin College, N.C. (USGS 02116500)</td>
<td>Davidson</td>
</tr>
<tr>
<td>A-II</td>
<td>Yadkin River at High Rock, N.C.</td>
<td>Davidson</td>
</tr>
<tr>
<td>&amp; B</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
(DEM, 1979). Solids from the Archie Elledge municipal treatment plant settle out during low flow (not an uncommon occurrence with wastewater treatment plant effluents). During storm flow conditions, the resuspension of organic matter has caused repeated fish kills down to the Yadkin River and High Rock Lake during the 1970's (DEM, 1979). These and other fish kills in the Yadkin River and some of its tributaries are discussed in further detail in a later section.

4.4 SALEM CREEK AND MUDDY CREEK

Selection of the Salem Creek and Muddy Creek river basins was based on a number of important considerations:

(1) a riverine environment was desired;

(2) a variety of point (industrial, municipal) and nonpoint (urban, agricultural runoff) sources contributed pollutants;

(3) a documented history of, and future potential for, adverse water quality impacts exists; and

(4) data availability, in spite of limitations thereof, was crucial.

A major impoundment, W. Kerr Scott Dam and Reservoir, ruled out much of the Upper Yadkin River. Dams virtually eliminate severe extremes in flow, both high and low, and incorporation of variability in water quantity is important because of its interactive role with water quality. Similarly, the lower Yadkin was ruled out. Downstream from High Rock Lake (east of Salisbury, North Carolina), the entire stretch of the Yadkin River south to the North Carolina-South Carolina state border is controlled by dams (refer to Figure IV-2). In downstream order, the reservoirs are: Tuckertown Lake (not shown), Badin Lake, Lake Tillery and Blewett Falls Lake.

**Land Use**

Winston-Salem and Forsyth County represent an area of widely varying land uses. Most of the City of Winston-Salem is drained by Salem Creek. It was noted in Chapter III that pollutant accumulation and washoff are functions of land use [refer to equation (3.27)]. Thus, an accurate estimate of land use types and percentages is essential. The results of a computerized analysis of the Salem Creek drainage area (downstream from Salem Lake) are presented in Figure IV-4 and Table IV-5, performed by the Land Resources Information Service (LRIS), North Carolina Department of Natural Resources and Community Development, Raleigh.

The application of such data for water quality modeling purposes
Figure IV-4. Land Resources Information Service Land Use Analysis for the Salem Creek Drainage Area
TABLE IV-5. Land Resources Information Service (LRIS) Land Use Analysis for Salem Creek Drainage Basin (LRIS, 1981)

<table>
<thead>
<tr>
<th>Land Use Type</th>
<th>Total Area, acres (hectares)</th>
<th>Percentage (%) of Total Area</th>
</tr>
</thead>
<tbody>
<tr>
<td>Residential</td>
<td>12,149.76 (4917)</td>
<td>42.93</td>
</tr>
<tr>
<td>Commercial</td>
<td>2,665.54 (1079)</td>
<td>9.42</td>
</tr>
<tr>
<td>Industrial</td>
<td>625.09 (253)</td>
<td>2.21</td>
</tr>
<tr>
<td>Roads</td>
<td>1,729.75 (700)</td>
<td>6.11</td>
</tr>
<tr>
<td>Other Urban</td>
<td>821.46 (332)</td>
<td>2.90</td>
</tr>
<tr>
<td>Cropland</td>
<td>3,253.96 (1317)</td>
<td>11.50</td>
</tr>
<tr>
<td>Deciduous Forest</td>
<td>6,033.73 (2442)</td>
<td>21.32</td>
</tr>
<tr>
<td>Evergreen Forest</td>
<td>276.88 (112)</td>
<td>0.98</td>
</tr>
<tr>
<td>Mixed Forest</td>
<td>628.72 (254)</td>
<td>2.22</td>
</tr>
<tr>
<td>Quarries</td>
<td>51.15 (21)</td>
<td>0.18</td>
</tr>
<tr>
<td>Transition</td>
<td>62.26 (25)</td>
<td>0.22</td>
</tr>
</tbody>
</table>

Total: 28,298.16 (11,452) 100%
is discussed in Chapter V. Projected land-use information could of course be used to project future pollutant accumulation rates. Population density projections can similarly be used to determine the municipal wastewater flows and resultant pollutant loading rates. General land use type projections for 1990, 2000 and 2010 were estimated in the Level B Existing Plan (U.S. Water Resources Council, 1979) for the Upper Yadkin. The values are presented in Table IV-6, expressed in percentages, for comparison purposes.

Current Population and Projections

In 1960, the Piedmont had less than 60 percent of North Carolina's total population, yet contained about 70 percent of the State's urban population (Putnam, 1972): the Winston-Salem population was estimated at 132,913. The Winston-Salem subbasin (essentially the Salem and Muddy Creek basins) of the Yadkin-Pee Dee contained 31 percent of the total 1970 population of approximately 875,000 (Harned and Meyer, 1981); in other words, about 271,250 persons. Population projections for Forsyth County by the Winston-Salem Chamber of Commerce and the Level B Study staff are compared in Table IV-7. Using an average figure for the year 2000 population projection, a 35.5 percent increase is projected in the county: an equivalent increase for the city of Winston-Salem would yield a population of 180,110, or an increase of about 47,200 in just 30 years. Obviously, a major increase in water use and sewage effluent can be expected which will further stress the waste assimilative capacity of Salem Creek. In 1980, the City of Winston-Salem diverted an average of 24.8 ft$^3$/sec (0.70 m$^3$/sec) from Salem Lake (at the headwaters of Salem Creek) to meet its water supply needs (U.S. Geological Survey, 1981), and about 26 ft$^3$/sec (0.74 m$^3$/sec) from the Yadkin River. An average sewage effluent of 42 ft$^3$/sec (1.19 m$^3$/sec) was returned to Salem Creek.

Estimates from the Winston-Salem/Forsyth County City-County Planning Board (Maya, 1982) for change in population from 1975 to 1985 and from 1985 to the year 2000 are presented in Figures IV-5 and IV-6, respectively.

Rainfall and Evaporation Stations

To meet the requirements of an intensive study of the effect of urban development on floods in the Piedmont province of North Carolina, 18 recording rain gages were installed in the Winston-Salem area by the U.S. Geological Survey from 1964 to 1970 (Putnam, 1972). Most of these rain gages were installed at stream gaging stations. Recently, recording rain gages were installed in the Central Business District (CBD) and the Ardmore residential area of Winston-Salem in a cooperative arrangement between the U.S. Geological Survey and the U.S. Environmental Protection Agency, as part of the National Urban Runoff Program (NURP) studies. Aside from these specialized, relatively short-
TABLE IV-6. Land Use Projections for Upper Yadkin, % (modified after U. S. Water Resources Council, 1979)

<table>
<thead>
<tr>
<th>Land Use Type</th>
<th>Year</th>
<th>1990</th>
<th>2000</th>
<th>2010</th>
</tr>
</thead>
<tbody>
<tr>
<td>Forest</td>
<td></td>
<td>52.3%</td>
<td>52.1%</td>
<td>50.5%</td>
</tr>
<tr>
<td>Agriculture</td>
<td></td>
<td>31.8</td>
<td>31.2</td>
<td>29.7</td>
</tr>
<tr>
<td>Urban</td>
<td></td>
<td>13.1</td>
<td>14.5</td>
<td>15.8</td>
</tr>
<tr>
<td>Miscellaneous</td>
<td></td>
<td>2.8</td>
<td>2.9</td>
<td>3.9</td>
</tr>
</tbody>
</table>

TABLE IV-7. Population Projections for Forsyth County

<table>
<thead>
<tr>
<th>YEAR</th>
<th>SOURCE</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Winston-Salem Chamber of Commerce</td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td>1980</td>
<td>242,581</td>
</tr>
<tr>
<td>1985</td>
<td>251,496</td>
</tr>
<tr>
<td>1990</td>
<td>264,216</td>
</tr>
<tr>
<td>1995</td>
<td>276,664</td>
</tr>
<tr>
<td>2000</td>
<td>288,638</td>
</tr>
<tr>
<td>2010</td>
<td></td>
</tr>
</tbody>
</table>

1. The 1970 Census estimate was 215,100.
2. Personal communication, 1981.
4. The percent change from 1970 to 2010 is 46.1% increase.
Figure IV-5. Population Projection for Winston-Salem/Forsyth County, 1975-1985. (Maya, 1982)

Figure IV-6. Population Projection for Winston-Salem/Forsyth County, 1985-2000. (Maya, 1982)
duration efforts, data from only four rainfall stations are readily accessible through HISARS, as shown in Figure IV-7. Only one of the four stations, at the R. A. Thomas Winston-Salem Water Filtration Plant, is located within the Salem Creek Drainage basin. This gage records only daily precipitation to the nearest hundredth of an inch.

Hourly rainfall is recorded at Greensboro WSO AP (a first-order NOAA, National Weather Service airport station, no. 313630) to the nearest hundredth of an inch. Long-term records are available from the NOAA Environmental Data and Information Service, National Climactic Center, Asheville, North Carolina on magnetic tape (e.g., 30-year record). Hourly rainfall is also recorded at Yadkinville 6E (NOAA station no. 319675) and Lexington 7N (NOAA station no. 314975). However, the Yadkinville station records to the nearest tenth of an inch and the Lexington records were often cumulative, not discretized hourly or even daily. The nearest evaporation stations, at approximately the same latitude as that of the Salem Creek basin, were at: W. Kerr Scott Dam and Reservoir (NOAA station no. 319555) to the west, and Chapel Hill 2W (NOAA station no. 311677) to the east, also shown in Figure IV-7.

Streamflow and Water Quality Monitoring Stations

Flow is regulated to a minor extent at the headwaters of Salem Creek by a small dam at Salem Lake, shown in Figures IV-8 and IV-9. Salem Creek flows southwesterly in shallow, sandy bottom channels. The creek is spanned in Figure IV-10 by the U.S. Highway 52 bridge and a railroad bridge further downstream. It continues to flow very near the Central Business District (see Figure IV-11), where the NURP rain gages are located. Further downstream the setting is rural, with heavy vegetation along the banks until the creek reaches USGS streamflow station no. 02115856 (Figure IV-12), 2700 feet (820 m) upstream from the Archie Elledge Wastewater Treatment Plant (Figure IV-13) effluent outfall. Downstream from its confluence with Salem Creek, and 100 feet (30m) upstream from Cooper Road (Secondary Road 2995) is Muddy Creek USGS station no. 02115860 (Figure IV-14).

Fluorescent dye studies were conducted by the U.S. Geological Survey from July 14, 1971 to March 14, 1973 to simulate the transport of water soluble wastes from the Winston-Salem area to the Yadkin River, in response to several major fish kills attributed to such wastes during heavy rains in the late 1960's and early 1970's (Lindskov, 1974). The entire study reach of 41 miles (66 km) is shown in Figure IV-15. The dye was released under four different flow conditions as an instantaneous slug at the Archie Elledge sewage treatment plant on Salem Creek. Total travel time ranged from 28 hours during periods of high streamflow to about 47 hours at low-flow. During high-flow conditions (discharge of 8,650 ft³/sec or 245 m³/sec at Yadkin College gage) the Rhodamine dye dispersed laterally across the river within a few miles.
Rainfall Stations (NWS station no.)
1. Greensboro WSO AP (313630)
2. Lexington 7N (314975)
3. Yadkinville 6E (319675)
4. R. A. Thomas Water Filtration Plant (319527)

Evaporation Stations (NWS station no.)
5. Chapel Hill 2W (311677)
6. W. Kerr Scott Dam and Reservoir (319555)

Figure IV-7. Rainfall and Evaporation Stations Nearest Winston-Salem, North Carolina

STREAMFLOW AND WATER QUALITY MONITORING STATIONS

Figure IV-8. Streamflow and Water Quality Monitoring Stations, Salem Creek and Muddy Creek Drainage Basin
Figure IV-9. Salem Dam and Lake.
Figure IV-10. Salem Creek Flowing Under U.S. Highway 52 Bridge and Railroad Bridge.
Figure IV-11. NURP Rain Gages in the Central Business District (top) and Main Street Bridge.
Figure IV-12. USGS Streamflow Station No. 02115856, Salem Creek.
Figure IV-13. Salem Creek Near Archie Elledge Wastewater Treatment Plant, City of Winston-Salem.
Figure IV-14. USGS Streamflow Station No. 02115860, Muddy Creek.
Figure IV-15. USGS Fluorescent Dye Study Reach, Salem Creek to Yadkin River.
(Lindskov, 1974)
downstream from its confluence with Muddy Creek. Under low-flow conditions (discharge of 2,290 ft³/sec or 64.9 m³/sec at Yadkin College) lateral dispersion was still incomplete more than 10 miles (16 km) downstream from the confluence.

During the dye studies, channel width at median discharge ranged from 35 feet (11 m) for Salem Creek to about 250 feet (76 meters) for the power reach of the Yadkin River. Corresponding depths ranged from 1.2 feet (0.37 m) for Salem Creek to about 7 feet (2 m) for the Yadkin River. The top photograph in Figure IV-16 is a view of the Yadkin River upstream of the Robinhood Road bridge, east of Winston-Salem: the water-surface width was measured to be about 210 feet (64 m). The bottom photograph was taken at the Shalloford Road bridge a few miles downstream: the water-surface depth was about 350 feet (107 m). The water-surface width at the Yadkin College gage (see Figure IV-3) on June 9, 1981 was approximately 220 feet (67 m) in late afternoon. These values are presented to appreciate the scale in the photographs; for example, the water-surface width at the Muddy Creek station (Figure IV-14) was 40 feet (12.2 m) on the same day a few hours earlier. Figure IV-17 represents views of the Yadkin River near Winston-Salem.

A discussion of the nature of the data, the quantity, quality and adequacy for mathematical modeling purposes is presented in Chapter V of a previous report (Medina, 1982).

4.5 SYNOPTIC RAINFALL DATA ANALYSIS

A description of the physical characteristics of the storm events that can be expected to occur over the study site requires synoptic analysis of long-term records. The variables of interest are: storm volume, duration, intensity and time between events -- as well as their recurrence intervals. The identification of seasonal trends is important for both hydrologic and receiving water quality impacts. A magnetic tape containing hourly rainfall recorded to the nearest hundredth of an inch for the period 1948 to 1975 at the Greensboro WSO first-order station (NOAA station no. 313630, Figure IV-7) was obtained from the National Climactic Center, Asheville, North Carolina. The station is located about 18 miles (28.8 km) east of the study site, sufficiently close for synoptic review of rainfall characteristics.

The record was processed first by program RFREQ to view the shape of the annual frequency histograms. The average histogram for the 28-year record of hourly rainfall is presented in Figure IV-18. The frequency distribution for the year 1974 (Figure IV-19) is very close. Thus, autocorrelation analysis of the 1974 hourly rainfall time series was performed by the modified version of SYNOP to define a minimum interevent time (see Section 3.1) of at least 11 consecutive hours of dry-weather to establish statistical storm event independence. The correlogram is presented in Figure IV-20. Hereafter,
Figure IV-16. Yadkin River at Robinhood Road (top) and the Shalloford Road Bridge (bottom), Winston-Salem.
Figure IV-17. Yadkin River Near Winston-Salem, North Carolina.
Figure VI-18. Average Annual Frequency Histogram of Hourly Rainfall, NOAA Station No. 313630

Figure IV-19. Frequency Histogram of Hourly Rainfall for 1974, NOAA Station No. 313630
CORRELOGRAM OF TIME SERIES, HOURLY RAINFALL, WINSTON-SALEM GREENSBORO AIRPORT

MINIMUM INTEREVENT TIME:
11 HOURS OF DRY WEATHER

NUMBER OF HOURLY LAGS

Figure IV-20. Correlogram of Hourly Rainfall for 1974,
NOAA Station No. 313630
the storm event analysis is based on the above value for minimum inter-event time. Figures IV-21 through IV-24 depict the distribution of storm depths (volume), duration, intensity and time since the previous storm—versus each month of the year (January through December). The graphs show clearly that high-intensity, short-duration storms occur during the summer (thunderstorms) while long-duration, low-intensity storms tend to occur in winter. While storm volumes can be expected to be higher in October, the fewest number of storms will occur during this month. Recurrence intervals (return period in years) are presented in Figures IV-25 through IV-28, respectively, for depth (volume), duration, average intensity and time since the previous storm.

Although periodicities are obvious, variations in storm variables from year to year do not exhibit the extreme deviations characteristic of the seasonal (monthly) grouping. The results are presented in Figures IV-29 through IV-32 for storm volume (rainfall depth), duration, rainfall intensity and time since the previous storm, respectively. Cumulative frequency curves for the same storm variables are shown in Figures IV-33 through IV-36, in the same order of presentation. It is interesting to note that, while storm volumes appear to deviate about a mean value near 0.5 inch (12.7 mm) in Figure IV-29, the probability of exceeding that value is only about 30 percent according to the curve in Figure IV-33. Similarly, the chance that a storm duration will exceed 24 hours is less than 10 percent, and there is a 50 percent chance that a storm duration of 6 hours will either be exceeded or not. It can be deduced from Figure IV-36 (and its tabular printout) that a 90 percent chance exists that the time since the last storm occurred will not exceed 202 hours, or that there is a 51 percent chance that the interevent time will be less than or equal to 72 hours.

4.6 FISH KILLS

Throughout this chapter, several references are made to major fish kills in Muddy Creek and the Upper Yadkin River, particularly in the segment downstream from its confluence with Muddy Creek to High Rock Lake. Official documentation is available for the incidents that occurred since the late 1960s and early 1970s, for example:


Figure IV-21. Storm Volumes Versus Month of Year, 1948-1975, NOAA Station No. 313630

Figure IV-22. Storm Duration Versus Month of Year, 1948-1975, NOAA Station No. 313630
Figure IV-23. Storm Intensities Versus Month of Year, 1948-1975, NOAA Station No. 313630

Figure IV-24. Time Since Previous Storm, 1948-1975, NOAA Station No. 313630
Figure IV-25. Recurrence Interval Versus Storm Volume, 1948-1975, NOAA Station No. 313630

Figure IV-26. Storm Duration Versus Recurrence Interval, 1948-1975, NOAA Station No. 313630
Figure IV-27. Storm Average Intensity Versus Recurrence Interval, 1948-1975, NOAA Station No. 313630

Figure IV-28. Recurrence Interval Versus Time Since Previous Storm, 1948-1975, NOAA Station No. 313630
Figure IV-29. Average Storm Depth in Each Year of Record, NOAA Station No. 313630

Figure IV-30. Average Duration in Each Year of Record, NOAA Station No. 313630
Figure IV-31. Average Rainfall Intensity in Each Year of Record, NOAA Station No. 313630

Figure IV-32. Average Time Since Previous Storm for Each Year of Record, NOAA Station No. 313630
Figure IV-33. Cumulative Frequency Curve for Rainfall Depth, 1948-1975, NOAA Station No. 313630

Figure IV-34. Cumulative Frequency Curve for Storm Duration, 1948-1975, NOAA Station No. 313630
Figure IV-35. Cumulative Frequency Curve for Average Rainfall Intensity, 1948-1975, NOAA Station No. 313630

Figure IV-36. Cumulative Frequency Curve for Time Since Previous Storm, 1948-1975, NOAA Station No. 313630
In addition, a large selection of newspaper articles on the subject, and video-taped live reports from the local television stations, are available. Perhaps one of the most spectacular (and most recent) of these fish kills occurred on June 26, 1981, when a mass of brewer's yeast leaked from a lagoon on a Davie County farm and spilled into the Yadkin River. The yeast, estimated at 1.4 million gallons, flowed 20 miles (32 km) downstream to High Rock Lake, consuming enough oxygen in the water to kill approximately 235,000 fish. The Pillsbury Company, Brakes, Inc. of Raleigh and the owner of a Davie County trucking company paid the State of North Carolina more than $61,000 in fines, but there was no evidence to indicate that the discharge was intentional. The fine included the cost of fish replacement, investigative costs and civil penalties for an illegal discharge resulting in water quality standards violations. The spill also adversely affected the quality of the City of Salisbury's water supply.

A sample of the headlines carried by area newspapers after the incident follows:

(1) "Massive fish kill reported on river," Salisbury Evening Post, June 26, 1981;

(2) "Dead fish for 25 miles city water gets extra chlorine," Salisbury Evening Post, June 27, 1981;

(3) "Salisbury officials discuss fish kill with Pillsbury Co." Salisbury Evening Post, July 9, 1981;
(4) "Yadkin River Fish Kill Reported," Durham Morning Herald, June 27, 1981;

(5) "Pollution Suspected in Yadkin Fish Kill," Durham Morning Herald, June 30, 1981;

(6) "Yadkin Fish Kill Brings Fine of Over $60,000" Durham Morning Herald, August 27, 1981;

(7) "Investigators Believe Pollution Killed Fish," Charlotte Observer, June 30, 1981;

(8) "Yadkin Fish Kill Is Probed," Winston-Salem Journal, June 27, 1981; and

(9) "Pollutant Linked To Death of Fish," Winston-Salem Journal, June 30, 1981.

A dramatic illustration of the severity of the incident is presented in Figures IV-37 and IV-38 (courtesy of the Salisbury Evening Post, June 26, 1981). The dissolved oxygen content at the reservoir's Yadkin River intake was as low as 1.7 mg/ℓ, where a normal reading of 7 mg/ℓ is expected during the summer. Among the dead fish were shad, carp, white bass, bream, lots of catfish, crappie and suckers.

The incident above, fortunately, is a rare occurrence which generated a lot of publicity. Fundamentally more serious, in terms of both historical frequency of occurrence and potential for recurrence, have been the fish kills attributed to hydrologic phenomena and human activity. In the summer of 1970, at least three major fish kills were blamed on poor operation of the Archie Elledge Wastewater Treatment Plant (Division of Environmental Management, 1979). A state board assessed damages at $23,202 (Salisbury Evening Post, June 26, 1981). Another major fish kill was reported on August 8, 1976, in Salem Creek, Muddy Creek, and in the Yadkin River from its confluence with Muddy Creek to its confluence with the South Yadkin River (Benton, 1976)--a total river stretch of 35.9 miles (57.4 km). Salem Creek was classified at the time as a "D" stream. Dissolved oxygen levels fell below 4.0 mg/ℓ for three consecutive hours (reaching zero for 45 minutes) at Yadkin College gage (refer to Figures IV-3 and IV-15), and DO levels did not reach 5.0 mg/ℓ for eight consecutive hours (Benton, 1976). According to the investigation:
Figure IV-37. A Workman Removes Dead Fish From the City of Salisbury’s Water Supply Reservoir, Near Ellis Crossroads, North Carolina. (Salisbury Evening Post, June 26, 1981)
Figure IV-38. A Closeup of Dead Fish Floating In the City of Salisbury Reservoir. (Salisbury Evening Post, June 26, 1981).
"Anaerobic solids had collected in Middle Fork Muddy Creek (Salem Creek) below the Archie Elledge Wastewater Treatment Plant before the rainstorms of 8-7-76. The storm flushed the solids from Middle Fork Muddy Creek into the Yadkin River and the resulting dissolved oxygen deficiency caused the Fish Kill."

The R. A. Thomas Water Filtration Plant rainfall station (NOAA Station No. 319675, Figure IV-7) in Winston-Salem recorded 0.31 inch (7.87 mm) on August 7th and 1.33 inches (33.8 mm) on August 8, 1976. The storm was preceded by 144 hours (6 days) of dry weather, and was followed by 120 hours (5 days) of zero precipitation -- clearly a statistically independent storm event by the analysis presented in Section 4.5. Furthermore, its total volume (rainfall depth of 1.64 inches), based on the 28-year record analyzed, has a return period of 2.5 months (Figure IV-25). The antecedent dry-weather period of 144 hours has a return period of 20 days (Figure IV-28). The biological investigation of the fish kill identified the following species: largemouth bass, sunfish, striped bass, white perch, white crappie, catfish, carp, suckers, and gizzard shad. Monetary damages (fish replacement, investigative costs) were assessed at over $9,000.

4.7 LOGNORMALITY OF FLOWS AND POLLUTANT CONCENTRATIONS

The dependency of the statistically-based models (presented in Chapter III) on the lognormality of flows and concentrations has been demonstrated. In Section 2.4, normal probability plots of the logarithmically transformed observations are presented for all nationwide sites of the NURP study. It is also of interest to examine the results obtained for the two NURP sites in Winston-Salem, North Carolina: the Central Business District and the Ardmore residential catchment. Figures IV-39 through IV-43 represent about 80 data points, and linearity is clearly visible. The results of fitting straight lines by linear regression analysis for transformed nonpoint, point and upstream flows and concentrations are presented in Chapter V.
Figure IV-39. Normal Probability Plots of Logarithms of Upstream Flows and Stormwater Surface Runoff, Winston-Salem NURP Sites

Figure IV-40. Normal Probability Plot of Logarithms of Stormwater Chemical Oxygen Demand Concentrations, Winston-Salem NURP Sites
Figure IV-41. Normal Probability Plots of Logarithms of Stormwater Total Suspended Solids and Total Phosphorus Concentrations, Winston-Salem NURP Sites

Figure IV-42. Normal Probability Plots of Logarithms of Stormwater Nitrogen Concentrations, Winston-Salem NURP Sites
Figure IV-43. Normal Probability Plots of Logarithms of Stormwater Zinc and Lead Concentrations, Winston-Salem NURP Sites
CHAPTER V
APPLICATIONS AND RESULTS

A detailed description of land use, population projections, climate, stream classifications and standards, fish species and fish kills, physical characteristics of storm events and other factors related to the hydrology and water quality of Salem Creek and Muddy Creek, North Carolina, is presented in Chapter IV. This chapter evaluates the results of hydrologic and water quality simulation and the adequacy of the data base for future modeling needs.

5.1 SOURCES OF CATCHMENT RESPONSE DATA

The collection of rainfall and evaporation data is reviewed in Chapter IV, and the location of stations is illustrated in Figure IV-7. Fortunately, accurate rainfall data were available because of N.C. Division of Environmental Management (DEM), in cooperation with the U.S. Geological Survey District Office, monitored rain gages in the Central Business District (CBD) and the Ardmore residential section, in Winston-Salem, N.C. Figure IV-8 illustrates the location of stream flow and water quality stations within the drainage basins, and is reproduced as Figure V-1 for convenience to the reader. The CBD rain gages are near the centroid of the Salem Creek drainage basin. Although available in smaller time increments, the rainfall data were averaged over 60-minute (hourly) time steps for the period of record from November 1980 to August 1981. Hourly streamflow was computed from measured gage height records-obtained directly from U.S. Geological Survey district and regional offices-by polynomial interpolation of rating curves.

Rating curves for stations No. 1 and No. 3 in Figure V-1 (USGS 02115856 and USGS 02115860, respectively) were available and have been discussed in great detail elsewhere (Medina, 1982). Station numbers reflected in Figure V-1 represent primary agency designations. For example, USGS 02115856 is also designated as Archie Elledge Wastewater Treatment Plant (WWTP) Upstream Monitoring Station No. NC0037834101. The WWTP discharge permit number is NC0037834, with three additional digits identifying monitoring locations: station no. 2 on the figure is WWTP Downstream Monitoring Station No. NC0037834104, while station no. 4 near Muddy Creek's confluence with the Yadkin River is WWTP Downstream Monitoring station no. NC0037834105. Station no. 3 (USGS 02115860, Muddy Creek near Muddy Creek, N.C.) is identified by the U.S. EPA STORET system as both YAD081 and NC030704008. Measurements of dissolved oxygen concentration and 5-day biochemical oxygen demand concentration are presented in Figures V-2 (Station no. 1, NC0037834101) and V-3 (station no. 2, NC0037834104).
Figure V-1. Streamflow and Water Quality Monitoring Stations, Salem Creek and Muddy Creek, North Carolina
Figure V-2. Archie Elledge WWTP Upstream Monitoring Station, Winston-Salem, NC

Figure V-3. Archie Elledge WWTP Downstream Monitoring Station, Winston-Salem, NC
5.2 LOGNORMALITY OF FLOWS AND CONCENTRATIONS, SALEM CREEK

Evidence has been presented in Chapters II and IV that nonpoint, point and upstream flows and concentrations in urban areas appear remarkably lognormally distributed. A set of microcomputer programs used to perform the statistical computations is described in Appendices A through K, including sample data sets and output. In particular, the programs presented in Appendices A and B are used to calculate the lognormal distribution and corresponding values of the standard normal variate — and then perform linear regression and plot the results.

Figures V-4 through V-9 represent the linear regression plots of the logarithmically transformed flows and BOD concentrations versus the standard normal deviate for, respectively: upstream, point (Archie Elledge WWTP effluent) and nonpoint sources, Salem Creek, Winston-Salem, North Carolina. The plots represent 727 data points for the time series November 1980 to August 1981. Values of the correlation coefficient squared (a measure of how much the variance of one variable is explained by its linear dependence on the other variable) ranged from 0.927 to 0.971. To insure that the programs are correct, the same data series were analyzed using the well-known SAS Institute procedures, and the results compared with those in Appendices C and F. An example of the job control language (JCL) required to execute the SAS UNIVARIATE procedure is presented in Section 6.1. Tables V-1 through V-6 list the SAS computed statistics. Figures V-10 through V-16 (except Figure V-14) represent the SAS normal probability plots of the logarithmically transformed variables versus the standard normal deviate.

5.3 HYDROLOGIC SIMULATION

To delineate watershed boundaries, USGS topographic maps of 1:250,000 scale (Charlotte and Winston-Salem) with contour intervals of 50 feet (15.2 m) were first used. Subsequently, 1:24,000 scale maps (Walkertown, Rural Hall, Winston-Salem East, Winston-Salem West, Belews Creek, and Kernersville) with contour intervals of 10 feet (3.05 m) were employed to provide a more accurate representation of slopes and areas. Land use information was obtained from the Land Resources Information Service (LRIS), described in Chapter IV. Impervious cover percentages were obtained from published data (Putnam, 1972). Discretization of Salem Creek and Muddy Creek into overland flow and stream flow segments is presented in Figures V-17 through V-19. The shaded area in Figure V-17 is the Salem Creek drainage basin, subjected to detailed land use analysis (see Figure IV-4 and Table IV-5). A reservoir segment (RS57) was used to simulate reservoir routing through Salem Lake. Stream segment SS58, flowing into Salem Lake, is essentially the headwaters of Salem Creek — known as Kerners Mill Creek above the dam. It should be noted that a finer discretization was in order for the Salem Creek drainage basin due to more rapidly varying topography and surface cover.
Figure V-4. Linear Regression of Log-Normally Distributed Upstream Flows, Salem Creek, near Winston-Salem, NC.

Figure V-5. Linear Regression of Log-Normally Distributed Upstream BOD Concentrations, Salem Creek, near Winston-Salem, NC.
Figure V-6. Linear Regression of Log-Normally Distributed Archie Elledge WWTP Effluent Flows, Salem Creek

Figure V-7. Linear Regression of Log-Normally Distributed Archie Elledge WWTP Effluent BOD Concentrations, Salem Creek
Figure V-8. Linear Regression of Log-Normally Distributed Storm and Point Flows, Salem Creek, near Winston-Salem, NC

Figure V-9. Linear Regression of Log-Normally Distributed Storm and Point BOD Concentrations, Salem Creek, near Winston-Salem, NC
TABLE V-1. SAS UNIVARIATE  
Statistics On Log-Upstream Flowx, Salem Creek,  
Winston-Salem, North Carolina

|               | SUM WGT$            | SUM WGT$            | VARIANCE | KURTOSIS | CSS | STD MEAN | PRCB>|T| | PRCB>|S| | <0.01 |
|---------------|---------------------|---------------------|----------|----------|-----|----------|------|------|------|------|
| N             | 727                 | 727                 | 2308.94  | 0.0805754| 0.936919 | 58.4977 | 0.0105277 | 0.0001 | 0.0001 | <0.01 |
| MEAN          | 3.17599             | SUM WGT$            | 727      | 2308.94  |       |          |       |      |      |      |
| STD DEV       | 0.283858            | VARIANCE            | 0.0805754| 0.936919 | 58.4977 | 0.0105277| 0.0001 | 0.0001 | <0.01 |
| SKEWNESS      | 0.679036            | KURTOSIS            | 0.936919 | 0.0805754| 0.936919 | 58.4977 | 0.0105277| 0.0001 | 0.0001 | <0.01 |
| U.S.S         | 739.167             | CSS                 | 58.4977  | 0.0105277| 0.0001 | <0.01   |       |      |      |      |
| CV            | 8.93763             | STD MEAN            | 0.0105277| 58.4977  | 0.0105277| 0.0001 | <0.01   |       |      |      |
| T:MEAN=0      | 301.679             | PRCB>|T|         | 0.0001   |          |       | <0.01   |       |      |      |      |
| SGN RANK      | 132314              | PRCB>|S|              | 0.0001   |          |       | <0.01   |       |      |      |
| NUM != 0      | 727                 | FROB>0             |          |          |       | <0.01   |       |      |      |
| D:NORMAL      | 0.0957004           | FROB>1             |          |          |       | <0.01   |       |      |      |

QUANTILES (DEF=4)

<table>
<thead>
<tr>
<th></th>
<th>100% MAX</th>
<th>99%</th>
<th>95%</th>
<th>90%</th>
<th>10%</th>
<th>5%</th>
<th>1%</th>
</tr>
</thead>
<tbody>
<tr>
<td>100% MAX</td>
<td>4.4391</td>
<td>3.9627</td>
<td>3.5946</td>
<td>3.5946</td>
<td>2.7912</td>
<td>2.7537</td>
<td>2.7147</td>
</tr>
<tr>
<td>75% Q3</td>
<td>3.3105</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>50% MED</td>
<td>3.1612</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>25% Q1</td>
<td>2.9957</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0% MIN</td>
<td>2.5416</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>RANGE</td>
<td>1.8975</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Q3-Q1</td>
<td>0.314799</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>MODE</td>
<td>2.9957</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

EXTREMES

<table>
<thead>
<tr>
<th>LOWEST</th>
<th>HIGHEST</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.5416</td>
<td>4.0307</td>
</tr>
<tr>
<td>2.5416</td>
<td>4.0775</td>
</tr>
<tr>
<td>2.5572</td>
<td>4.0943</td>
</tr>
<tr>
<td>2.5953</td>
<td>4.237</td>
</tr>
<tr>
<td>2.6672</td>
<td>4.4391</td>
</tr>
</tbody>
</table>

Note: See mean, standard deviation and skewness under maximum likelihood, page F-1.
TABLE V-2. SAS UNIVARIATE
Statistics On Log-Upstream BOD Concentrations, Salem Creek, Winston-Salem, North Carolina

<table>
<thead>
<tr>
<th>MOMENTS</th>
<th>SUM WEGTS</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>N</td>
<td>727</td>
<td></td>
</tr>
<tr>
<td>MEAN</td>
<td>1.2315</td>
<td></td>
</tr>
<tr>
<td>STD DEV</td>
<td>0.574343</td>
<td></td>
</tr>
<tr>
<td>SKEWNESS</td>
<td>-0.0726648</td>
<td></td>
</tr>
<tr>
<td>USS</td>
<td>1342.05</td>
<td></td>
</tr>
<tr>
<td>CV</td>
<td>46.6376</td>
<td></td>
</tr>
<tr>
<td>T:MEAN=0</td>
<td>57.8138</td>
<td></td>
</tr>
<tr>
<td>SGN RANK</td>
<td>128828</td>
<td></td>
</tr>
<tr>
<td>NUM -= 0</td>
<td>721</td>
<td></td>
</tr>
<tr>
<td>D: NORMAL</td>
<td>0.194315</td>
<td></td>
</tr>
<tr>
<td></td>
<td>SUM</td>
<td>895.302</td>
</tr>
<tr>
<td></td>
<td>VARIANCE</td>
<td>0.329869</td>
</tr>
<tr>
<td></td>
<td>KURTOSIS</td>
<td>2.54191</td>
</tr>
<tr>
<td></td>
<td>CSS</td>
<td>239.485</td>
</tr>
<tr>
<td></td>
<td>STD MEAN</td>
<td>0.0213012</td>
</tr>
<tr>
<td></td>
<td>PROB&gt;</td>
<td>T</td>
</tr>
<tr>
<td></td>
<td>PROB&gt;</td>
<td>S</td>
</tr>
<tr>
<td>QUANTILES(DEP=4)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>100% MAX</td>
<td>2.9444</td>
<td>99% 2.9444</td>
</tr>
<tr>
<td>75% Q3</td>
<td>1.3863</td>
<td>95% 2.15006</td>
</tr>
<tr>
<td>50% MED</td>
<td>1.0986</td>
<td>90% 1.9459</td>
</tr>
<tr>
<td>25% Q1</td>
<td>1.0986</td>
<td>10% 0.6931</td>
</tr>
<tr>
<td>0% MIN</td>
<td>-0.6931</td>
<td>5% 0.4055</td>
</tr>
<tr>
<td>RANGE</td>
<td>3.6375</td>
<td>1% -0.6931</td>
</tr>
<tr>
<td>Q3-Q1</td>
<td>0.2877</td>
<td></td>
</tr>
<tr>
<td>MODE</td>
<td>1.3863</td>
<td></td>
</tr>
</tbody>
</table>

| EXTREMES            |           |     |
| LOWEST             |            |     |
| -0.6931            |            |     |
| -0.6931            |            |     |
| -0.6931            |            |     |
| -0.6931            |            |     |
| -0.6931            |            |     |
| HIGHEST            |            |     |
| 2.9444             |            |     |
| 2.9444             |            |     |
| 2.9444             |            |     |
| 2.9444             |            |     |
| 2.9444             |            |     |

Note: See mean, standard deviation and skewness under maximum likelihood, page F-2.
TABLE V-3. SAS UNIVARIATE
Statistics On Log-Storm
and Point Flows, Salem Creek,
Winston-Salem, North Carolina

<table>
<thead>
<tr>
<th></th>
<th>N</th>
<th>SUM WGTS</th>
<th></th>
<th>N</th>
<th>SUM</th>
</tr>
</thead>
<tbody>
<tr>
<td>MEAN</td>
<td>4.55768</td>
<td>727</td>
<td>SUM WGT</td>
<td>727</td>
<td>3313.43</td>
</tr>
<tr>
<td>STD DEV</td>
<td>0.757276</td>
<td>100%</td>
<td>VARIANCE</td>
<td>0.573466</td>
<td></td>
</tr>
<tr>
<td>SKEWNESS</td>
<td>0.430385</td>
<td>93%</td>
<td>KURTOSIS</td>
<td>-0.684661</td>
<td></td>
</tr>
<tr>
<td>USS</td>
<td>1551.79</td>
<td>50%</td>
<td>CSS</td>
<td>416.337</td>
<td></td>
</tr>
<tr>
<td>CV</td>
<td>16.6154</td>
<td>25%</td>
<td>STD MEAN</td>
<td>0.0280858</td>
<td></td>
</tr>
<tr>
<td>T: MEAN = 0</td>
<td>162.277</td>
<td>Q1</td>
<td>PROB &gt;</td>
<td>T</td>
<td></td>
</tr>
<tr>
<td>SGN RANK</td>
<td>132314</td>
<td>Q3</td>
<td>PROB &gt;</td>
<td>S</td>
<td></td>
</tr>
<tr>
<td>NUM -= 0</td>
<td>727</td>
<td>MED</td>
<td>FRCE &gt; D</td>
<td>&lt;0.01</td>
<td></td>
</tr>
<tr>
<td>D: NORMAL</td>
<td>0.0731503</td>
<td>RANGE</td>
<td>FRCE &gt; D</td>
<td>&lt;0.01</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Q3 - Q1</td>
<td></td>
<td></td>
<td>MODE</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1.0648</td>
<td></td>
<td></td>
<td>3.6968</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3.5503</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>100% MAX</td>
<td>6.474</td>
<td>99%</td>
<td>6.26108</td>
<td>6.3353</td>
<td></td>
</tr>
<tr>
<td>75% Q3</td>
<td>5.0709</td>
<td>95%</td>
<td>5.95468</td>
<td>6.3551</td>
<td></td>
</tr>
<tr>
<td>50% MED</td>
<td>4.4209</td>
<td>90%</td>
<td>5.7232</td>
<td>6.4207</td>
<td></td>
</tr>
<tr>
<td>25% Q1</td>
<td>4.0061</td>
<td>10%</td>
<td>3.6168</td>
<td>6.4272</td>
<td></td>
</tr>
<tr>
<td>0% MIN</td>
<td>2.9237</td>
<td>5%</td>
<td>3.5181</td>
<td>6.474</td>
<td></td>
</tr>
</tbody>
</table>

QUANTILES (DEF=4)

EXTREMES

Note: See mean, standard deviation and skewness .
under maximum likelihood, page F-3.
TABLE V-4. SAS UNIVARIATE
Statistics On Log-Storm
and Point BOD Concentrations
Salem Creek, Winston-Salem,
North Carolina

<table>
<thead>
<tr>
<th>MOMENTS</th>
<th>SUM WGT$S$</th>
<th>SUM WGT$S$</th>
</tr>
</thead>
<tbody>
<tr>
<td>N</td>
<td>727</td>
<td>727</td>
</tr>
<tr>
<td>MEAN</td>
<td>3.38666</td>
<td>2462.1</td>
</tr>
<tr>
<td>STD DEV</td>
<td>0.70619</td>
<td>VARIANCE</td>
</tr>
<tr>
<td>SKEWNESS</td>
<td>-0.560661</td>
<td>0.498705</td>
</tr>
<tr>
<td>U.S.</td>
<td>8709.36</td>
<td>KURTOSIS</td>
</tr>
<tr>
<td>CV</td>
<td>20.8521</td>
<td>CSS</td>
</tr>
<tr>
<td>T: MEAN=0</td>
<td>129.305</td>
<td>STD MEAN</td>
</tr>
<tr>
<td>SGN RANK</td>
<td>132314</td>
<td>0.0261912</td>
</tr>
<tr>
<td>NUM =: 0</td>
<td>727</td>
<td>PROB&gt;</td>
</tr>
<tr>
<td>D: NORMAL</td>
<td>0.0962048</td>
<td>PROB&gt;</td>
</tr>
<tr>
<td></td>
<td></td>
<td>&lt;0.01</td>
</tr>
</tbody>
</table>

QUANTILES(DEF=4)

<table>
<thead>
<tr>
<th>QUANTILES(DEF=4)</th>
<th>100% MAX</th>
<th>99%</th>
<th>4.85775</th>
</tr>
</thead>
<tbody>
<tr>
<td>75% Q3</td>
<td>3.8135</td>
<td>95%</td>
<td>4.51546</td>
</tr>
<tr>
<td>50% MED</td>
<td>3.4965</td>
<td>90%</td>
<td>4.24734</td>
</tr>
<tr>
<td>25% Q1</td>
<td>3.0364</td>
<td>10%</td>
<td>2.39754</td>
</tr>
<tr>
<td>0% MIN</td>
<td>0.6931</td>
<td>5%</td>
<td>2.0794</td>
</tr>
<tr>
<td>RANGE</td>
<td>4.3257</td>
<td>1%</td>
<td>1.3863</td>
</tr>
<tr>
<td>Q3-Q1</td>
<td>0.7771</td>
<td></td>
<td></td>
</tr>
<tr>
<td>MODE</td>
<td>2.0794</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

EXTREMES

<table>
<thead>
<tr>
<th>EXTREMES</th>
<th>LOWEST</th>
<th>HIGHEST</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.6931</td>
<td>4.927</td>
<td></td>
</tr>
<tr>
<td>0.6931</td>
<td>4.967</td>
<td></td>
</tr>
<tr>
<td>1.0986</td>
<td>4.9988</td>
<td></td>
</tr>
<tr>
<td>1.0986</td>
<td>4.9993</td>
<td></td>
</tr>
<tr>
<td>1.3863</td>
<td>5.0188</td>
<td></td>
</tr>
</tbody>
</table>

Note: See mean, standard deviation and skewness under maximum likelihood, page F-4.
## TABLE V-5. SAS UNIVARIATE

Statistics On Log-Point Source Flows, Salem Creek, Winston-Salem, North Carolina

### MOMENTS

<p>| | | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>N</td>
<td>727</td>
<td>SUM WGTS</td>
</tr>
<tr>
<td>MEAN</td>
<td>3.96084</td>
<td>SUM</td>
</tr>
<tr>
<td>STD DEV</td>
<td>0.306025</td>
<td>VARIANCE</td>
</tr>
<tr>
<td>SKEWNESS</td>
<td>-0.254497</td>
<td>KURTOSIS</td>
</tr>
<tr>
<td>USS</td>
<td>11473.4</td>
<td>CSS</td>
</tr>
<tr>
<td>CV</td>
<td>7.72627</td>
<td>STD MEAN</td>
</tr>
<tr>
<td>T:MEAN=0</td>
<td>348.977</td>
<td>FRGB&gt;T</td>
</tr>
<tr>
<td>SGN RANK</td>
<td>132314</td>
<td>FRGB&gt;S</td>
</tr>
<tr>
<td>NUM -&gt; Q</td>
<td>727</td>
<td></td>
</tr>
<tr>
<td>D:NORMAL</td>
<td>0.144255</td>
<td></td>
</tr>
</tbody>
</table>

### QUANTILES (DEF=4)

<table>
<thead>
<tr>
<th></th>
<th>100% MAX</th>
<th>75% Q3</th>
<th>50% MED</th>
<th>25% Q1</th>
<th>0% MIN</th>
</tr>
</thead>
<tbody>
<tr>
<td>LOWEST</td>
<td>2.9237</td>
<td>3.2112</td>
<td>3.2722</td>
<td>3.3293</td>
<td>3.3293</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th></th>
<th>RANGE</th>
<th>Q3-Q1</th>
<th>MODE</th>
</tr>
</thead>
<tbody>
<tr>
<td>LOWEST</td>
<td>1.554</td>
<td>0.5587</td>
<td>3.6968</td>
</tr>
<tr>
<td>HIGHEST</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### EXTREMES

<table>
<thead>
<tr>
<th>LOWEST</th>
<th>2.9237</th>
<th>4.4777</th>
</tr>
</thead>
<tbody>
<tr>
<td>HIGHEST</td>
<td>4.4777</td>
<td>4.4777</td>
</tr>
</tbody>
</table>

Note: See mean, standard deviation and skewness under maximum likelihood, page F-5.
TABLE V-6. SAS UNIVARIATE
Statistics On Log-Point Source BOD Concentrations,
Salem Creek, Winston-Salem,
North Carolina

Moments

<p>| | | | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>N</td>
<td>727</td>
<td>SUM WGT</td>
<td>727</td>
</tr>
<tr>
<td>MEAN</td>
<td>2.90225</td>
<td>SUM</td>
<td>2109.94</td>
</tr>
<tr>
<td>STD DEV</td>
<td>0.744058</td>
<td>VARIANCE</td>
<td>0.553622</td>
</tr>
<tr>
<td>SKEWNESS</td>
<td>-0.178723</td>
<td>KURTOSIS</td>
<td>-0.382724</td>
</tr>
<tr>
<td>USS</td>
<td>652.5</td>
<td>CSS</td>
<td>401.929</td>
</tr>
<tr>
<td>CV</td>
<td>25.6372</td>
<td>STD MEAN</td>
<td>0.0275956</td>
</tr>
<tr>
<td>T:MEAN=0</td>
<td>105.171</td>
<td>PRCB&gt;T</td>
<td>0.0001</td>
</tr>
<tr>
<td>SGN RANK</td>
<td>132314</td>
<td>PRCB&gt;S</td>
<td>0.0001</td>
</tr>
<tr>
<td>NUM -= 0</td>
<td>727</td>
<td></td>
<td></td>
</tr>
<tr>
<td>D: NORMAL</td>
<td>0.102629</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Quantiles (DEF=4)

<p>| | | | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>100% MAX</td>
<td>4.7707</td>
<td>99%</td>
<td>4.7707</td>
</tr>
<tr>
<td>75% Q3</td>
<td>3.4657</td>
<td>95%</td>
<td>3.9703</td>
</tr>
<tr>
<td>50% MED</td>
<td>3.0445</td>
<td>90%</td>
<td>3.8286</td>
</tr>
<tr>
<td>25% Q1</td>
<td>2.3026</td>
<td>10%</td>
<td>1.7918</td>
</tr>
<tr>
<td>0% MIN</td>
<td>0.6931</td>
<td>5%</td>
<td>1.7918</td>
</tr>
<tr>
<td>RANGE</td>
<td>4.0776</td>
<td>1%</td>
<td>1.3863</td>
</tr>
<tr>
<td>Q3-Q1</td>
<td>1.1631</td>
<td></td>
<td></td>
</tr>
<tr>
<td>MODE</td>
<td>1.7918</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Extremes

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>LOWEST</td>
<td>0.6931</td>
</tr>
<tr>
<td>0.6931</td>
<td></td>
</tr>
<tr>
<td>1.0986</td>
<td></td>
</tr>
<tr>
<td>1.0986</td>
<td></td>
</tr>
<tr>
<td>1.3863</td>
<td></td>
</tr>
<tr>
<td>HIGHEST</td>
<td>4.7707</td>
</tr>
<tr>
<td>4.7707</td>
<td></td>
</tr>
<tr>
<td>4.7707</td>
<td></td>
</tr>
<tr>
<td>4.7707</td>
<td></td>
</tr>
<tr>
<td>4.7707</td>
<td></td>
</tr>
</tbody>
</table>

Note: See mean, standard deviation and skewness under maximum likelihood, page F-6.
Figure V-10. SAS UNIVARIATE Normal Probability Plot on Log-Upstream Flows, Salem Creek

Figure V-11. SAS UNIVARIATE Normal Probability Plot on Log-Upstream BOD Concentrations, Salem Creek
Figure V-12. SAS UNIVARIATE Normal Probability Plot on Log-Storm and Point Flows, Salem Creek

UNIVARIATE on LOG(X(I))
SALEM CREEK, WINSTON-SALEM
NORTH CAROLINA

Figure V-13. SAS UNIVARIATE Normal Probability Plot on Log-Storm and Point Source BOD Concentrations, Salem Creek
Figure V-14. SAS UNIVARIATE Frequency Distribution Bar Chart on Log-Storm and Point Source BOD Concentrations, Salem Creek

Figure V-15. SAS UNIVARIATE Normal Probability Plot on Log-Point Source Flows, Salem Creek
Figure V-16. SAS UNIVARIATE Normal Probability Plot on Log-Point Source BOD Concentrations, Salem Creek

Figure V-17. Muddy Creek River Basin Discretization for Hydraulic Stimulation
Figure V-18. Discretization of Salem Creek (Kerners Mill Creek) Above Salem Lake and Dam

Figure V-19. Discretization of Salem Creek into Overland Flow and Stream Flow Segments
A manual planimeter was used to obtain all sub-basin areas; however, these values were checked by electronic planimeter: by U.S. Geological Survey Raleigh District Office personnel and the computerized LRIS system. For example, the manually obtained area for Salem Creek downstream from Salem Dam was within less than 2% of the value obtained by LRIS. Table V-7 summarizes overland flow segment areas for the Salem Creek drainage basin above and below Salem Dam and Lake. The results of model calibration are presented in Figure V-20. The simulated peak is 298.47 ft³/sec (8.45 cu m/sec), while the peak discharge was measured to be 297.70 ft³/sec (8.43 cu m/sec), within 0.26 percent error. The simulated volume of rainfall excess is 0.113 inches (287 mm) and the measured direct runoff 0.111 inches (2.82 mm), within 1.8 percent error. As a matter of interest, the measured rainfall volume was 0.71 inches (18.0 mm) such that the coefficient of runoff would be about 0.16. Verification is presented in Figure V-21. Measured and simulated peak discharges are, respectively: 605 ft³/sec (17.1 cu m/sec) and 502 ft³/sec (14.2 cu m/sec), within 17 percent error. The measured rainfall excess and simulated runoff are, respectively: 0.189 inches (4.8 mm) and 0.217 inches (5.5 mm), within 15 percent error. The measured rainfall volume for the February 10, 1981 storm is 1.29 inches (32.8 mm), for a coefficient of runoff of approximately 0.15. The complete runoff time series was stored in a disk file for use in the deterministic water quality simulations.

5.4 WATER QUALITY SIMULATION

In Chapter I, a hierarchical approach for instream water quality analysis is proposed (see Table I-1). The preliminary screening analysis is required before any further computation can be contemplated, and the results of that exercise are largely presented in Chapter IV for the study site. The results of applying Levels II, III and IV in an integrated manner are presented in this section. The unifying theme of the three levels is the concept of water quality frequency curves, discussed in Chapter II.

In Chapter IV, North Carolina stream classifications and standards were reviewed. In particular, stream segments within the Yadkin River basin in North Carolina were identified. Kerners Mill Creek (Salem Creek above Salem Lake and Dam) is classified A-II: source of water supply for drinking and other purposes specified by the C classification. The remaining segment of Salem Creek downstream from the dam, and the entire Muddy Creek segment from its source to the Yadkin River, are classified as C: fish and wildlife propagation and other uses. As non-trout waters the minimum required dissolved oxygen concentration is 5.0 mg/L. It has been noted; however, that the stream segment of Salem Creek (Middle Fork Muddy Creek) from Bushy Fork to Muddy Creek is not considered satisfactory for boating or wading usage (DEM, June 1981) due to exceedence of fecal coliform bacteria levels for such activity.
### TABLE V-7. Overland Flow Segment Areas for Salem Creek Above and Below Salem Dam and Lake

<table>
<thead>
<tr>
<th>Overland Flow Section</th>
<th>Area, acres (hectares)</th>
</tr>
</thead>
<tbody>
<tr>
<td>OF01</td>
<td>880 (356)</td>
</tr>
<tr>
<td>OF02</td>
<td>780 (316)</td>
</tr>
<tr>
<td>OF03</td>
<td>1400 (567)</td>
</tr>
<tr>
<td>OF04</td>
<td>1730 (700)</td>
</tr>
<tr>
<td>OF05</td>
<td>1050 (425)</td>
</tr>
<tr>
<td>OF06</td>
<td>930 (376)</td>
</tr>
<tr>
<td>OF07</td>
<td>850 (344)</td>
</tr>
<tr>
<td>OF08</td>
<td>2680 (1085)</td>
</tr>
<tr>
<td>OF09</td>
<td>1590 (643)</td>
</tr>
<tr>
<td>OF10</td>
<td>280 (113)</td>
</tr>
<tr>
<td>OF11</td>
<td>800 (324)</td>
</tr>
<tr>
<td>OF12</td>
<td>300 (121)</td>
</tr>
<tr>
<td>OF13</td>
<td>1020 (413)</td>
</tr>
<tr>
<td>OF14</td>
<td>1130 (457)</td>
</tr>
<tr>
<td>OF15</td>
<td>1390 (563)</td>
</tr>
<tr>
<td>OF16</td>
<td>520 (210)</td>
</tr>
<tr>
<td>OF17</td>
<td>900 (364)</td>
</tr>
<tr>
<td>OF18</td>
<td>430 (174)</td>
</tr>
<tr>
<td>OF19</td>
<td>300 (121)</td>
</tr>
<tr>
<td>OF20</td>
<td>190 (77)</td>
</tr>
<tr>
<td>OF21</td>
<td>1200 (486)</td>
</tr>
<tr>
<td>OF22</td>
<td>1050 (425)</td>
</tr>
<tr>
<td>OF23</td>
<td>880 (356)</td>
</tr>
<tr>
<td>OF24</td>
<td>630 (255)</td>
</tr>
<tr>
<td>OF25</td>
<td>530 (214)</td>
</tr>
<tr>
<td>OF26</td>
<td>1780 (720)</td>
</tr>
<tr>
<td>OF27</td>
<td>740 (299)</td>
</tr>
<tr>
<td>OF28</td>
<td>930 (376)</td>
</tr>
<tr>
<td>OF29</td>
<td>410 (166)</td>
</tr>
<tr>
<td>OF30</td>
<td>490 (198)</td>
</tr>
<tr>
<td><strong>Subtotal: Downstream</strong></td>
<td><strong>27790 (11247)</strong></td>
</tr>
<tr>
<td><strong>of Salem Dam</strong></td>
<td></td>
</tr>
<tr>
<td>OF55</td>
<td>8000 (3238)</td>
</tr>
<tr>
<td>OF56</td>
<td>8860 (3586)</td>
</tr>
<tr>
<td><strong>Subtotal: Upstream</strong></td>
<td><strong>16860 (6823)</strong></td>
</tr>
<tr>
<td><strong>of Salem Dam</strong></td>
<td></td>
</tr>
<tr>
<td><strong>Salem Lake</strong></td>
<td><strong>400 (162)</strong></td>
</tr>
<tr>
<td><strong>Total Basin:</strong></td>
<td><strong>45050 (18332)</strong></td>
</tr>
</tbody>
</table>
The stream fecal coliform standard for class C receiving waters is 1000 MPN/100 ml.

The two statistically-based receiving water quality frequency distribution models, the moments approximation model and the Gaussian quadrature model, have been presented in Chapter III. The results of predicting the frequency distribution of BOD concentrations in Salem Creek, for the time series November 1980 to August 1981, are shown in Figure V-22. The predictions are almost identical; however, execution times of the moments approximation model are much faster than those of the Gaussian quadrature model. The simpler moments approximation model was therefore selected for the other simulations: Figure V-23 represents the predicted frequency curve for chemical oxygen demand (COD) concentrations in Salem Creek during dry weather conditions; and Figure V-24 represents the predicted frequency curve for fecal coliform concentrations in Salem Creek, for the same time series. These results indicate that the stream fecal coliform standard was exceeded (violated) approximately 48 percent of the time during dry weather flow conditions. Predicted output for both of these two simulations is presented in Tables V-9 and V-10. Both Figures V-23 and V-24 are computer-generated plots (see Appendix H).

Pollutant accumulation rates on the land surface are highly dependent upon land use and must be specified for deterministic simulation. STORM was used to generate watershed pollutant washoff loading rates. Suggested pollutant accumulation rates are reviewed in Table V-11, as recommended by the Hydrologic Engineering Center (1977) and the Triangle J Council of Governments (1976). The commercial and industrial rates were found to be too high for those land uses in the Winston-Salem area. Interfacing the output from STORM with both deterministic models, STO/TRT RECEIVING and LEVEL III - RECEIVING, is discussed in Section 6.1, Chapter VI. Calibration to measured data has been discussed in a previous report (Medina, 1982). Model parameters such as deoxygenation rate were adjusted, while reaeration rate and longitudinal dispersion were calculated internally according to the methodology presented in Chapter III. Advective and dispersive transport by STO/TRT RECEIVING was in close agreement with time-of-travel measurements conducted in Salem Creek and Muddy Creek by the U.S. Geological Survey (Lindskov, 1974) and the N.C. Division of Environmental Management (Buzun, 1982). Most of the data from the latter intensive survey was collected during 1976 as part of waste load allocation studies. A summary of key model parameters for both LEVEL III - RECEIVING and STO/TRT RECEIVING is presented in Table V-12. Both models are compared in terms of predicted cumulative dissolved oxygen concentration frequency curves in Figure V-25. The steady-state model overpredicts the impact on DO levels up to about the stream standard, beyond which it slightly underpredicts the oxygen deficit. It is also of interest to compare the performance of the steady-state deterministic model versus that of the statistical (moments approximation) model, presented in Figure V-26. The results are
OUTFLOW HYDROGRAPH CALIBRATION
STORM EVENT NO.3 11/17/80
SALEM CREEK NEAR ATWOOD, N.C.

- Observed
- Simulated

Figure V-20. Salem Creek Calibration, USGS Distributed Routing Rainfall-Runoff Model

OUTFLOW HYDROGRAPH VERIFICATION
STORM EVENT NO.8 2/10/81
SALEM CREEK NEAR ATWOOD, N.C.

- Observed
- Simulated

Figure V-21. Salem Creek Verification, USGS Distributed Routing Rainfall-Runoff Model
Figure V-22. Predicted Stream BOD Concentration Frequency Distribution, Moments Approximation and Gaussian Quadrature

Figure V-23. Predicted COD Concentration Frequency Distribution, Salem Creek, November 1980 to August 1981
Figure V-24. Predicted Fecal Coliform Bacteria Concentration Frequency Distribution, Salem Creek, Nov. 1980 to Aug. 1981
TABLE V-9. Predicted Output From Moments Approximation Model, COD, Salem Creek

<table>
<thead>
<tr>
<th>STREAM CONCENTRATION (CO)</th>
<th>PROBABILITY AND RETURN PERIOD</th>
</tr>
</thead>
<tbody>
<tr>
<td>Runoff Loads</td>
<td>(MOMENTS METHOD)</td>
</tr>
<tr>
<td>---------------------------</td>
<td>--------------------------------</td>
</tr>
</tbody>
</table>

| COD | 29 Storms/Year | RHO 0 |

<table>
<thead>
<tr>
<th>===========&gt;</th>
<th>MEAN</th>
<th>C VAR</th>
</tr>
</thead>
<tbody>
<tr>
<td>Upstr Conc CS</td>
<td>14.04</td>
<td>0.904</td>
</tr>
<tr>
<td>Upstr Flow DS</td>
<td>25.00</td>
<td>0.324</td>
</tr>
<tr>
<td>Storm Conc CR</td>
<td>139.47</td>
<td>0.425</td>
</tr>
<tr>
<td>Dnstr Conc CO</td>
<td>96.85</td>
<td>0.426</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Stream Conc CO</th>
<th>% Events Which Exceed</th>
<th>Return Period (Yrs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>35.00</td>
<td>98.886</td>
<td>0.0349</td>
</tr>
<tr>
<td>40.00</td>
<td>97.493</td>
<td>0.0354</td>
</tr>
<tr>
<td>45.00</td>
<td>95.261</td>
<td>0.0362</td>
</tr>
<tr>
<td>50.00</td>
<td>92.116</td>
<td>0.0374</td>
</tr>
<tr>
<td>55.00</td>
<td>88.095</td>
<td>0.0391</td>
</tr>
<tr>
<td>60.00</td>
<td>83.321</td>
<td>0.0414</td>
</tr>
<tr>
<td>65.00</td>
<td>77.988</td>
<td>0.0442</td>
</tr>
<tr>
<td>70.00</td>
<td>72.235</td>
<td>0.0477</td>
</tr>
<tr>
<td>75.00</td>
<td>66.315</td>
<td>0.0520</td>
</tr>
<tr>
<td>80.00</td>
<td>60.380</td>
<td>0.0571</td>
</tr>
<tr>
<td>85.00</td>
<td>54.574</td>
<td>0.0632</td>
</tr>
<tr>
<td>90.00</td>
<td>49.006</td>
<td>0.0704</td>
</tr>
<tr>
<td>95.00</td>
<td>43.755</td>
<td>0.0788</td>
</tr>
<tr>
<td>100.00</td>
<td>38.872</td>
<td>0.0887</td>
</tr>
<tr>
<td>105.00</td>
<td>34.384</td>
<td>0.1003</td>
</tr>
<tr>
<td>110.00</td>
<td>30.299</td>
<td>0.1138</td>
</tr>
<tr>
<td>115.00</td>
<td>26.614</td>
<td>0.1296</td>
</tr>
<tr>
<td>120.00</td>
<td>23.311</td>
<td>0.1479</td>
</tr>
<tr>
<td>STREAM CONCENTRATION (CO)</td>
<td>PROBABILITY AND RETURN PERIOD</td>
<td>RUNOFF LOADS</td>
</tr>
<tr>
<td>---------------------------</td>
<td>-------------------------------</td>
<td>--------------</td>
</tr>
<tr>
<td>(MOMENTS METHOD)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>--------------------------</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**TABLE V-10. Predicted Output From Moments Approximation Model, Fecal Coliform, Salem Creek**

<table>
<thead>
<tr>
<th>FCOlI</th>
<th>29 STORMS/YEAR</th>
<th>RHO 0</th>
</tr>
</thead>
</table>

| STREAM CONC CS | 2657.30 | 1.649 |
| UPSTR FLOW CS  | 25.00  | 0.324 |
| STORM FLOW QR  | 54.93  | 0.292 |
| STORM CONC CR  | 3301.10| 4.180 |
| DNSTR CONC CO  | 3082.35| 3.024 |

<table>
<thead>
<tr>
<th>STREAM CONC (CO)</th>
<th>% EVENTS</th>
<th>RETURN</th>
<th>PERIOD</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>(YRS)</td>
</tr>
<tr>
<td>30.00</td>
<td>98.876</td>
<td>0.0349</td>
<td></td>
</tr>
<tr>
<td>280.00</td>
<td>79.237</td>
<td>0.0435</td>
<td></td>
</tr>
<tr>
<td>530.00</td>
<td>65.376</td>
<td>0.0527</td>
<td></td>
</tr>
<tr>
<td>780.00</td>
<td>55.631</td>
<td>0.0620</td>
<td></td>
</tr>
<tr>
<td>1030.00</td>
<td>48.364</td>
<td>0.0713</td>
<td></td>
</tr>
<tr>
<td>1280.00</td>
<td>42.710</td>
<td>0.0807</td>
<td></td>
</tr>
<tr>
<td>1530.00</td>
<td>38.172</td>
<td>0.0903</td>
<td></td>
</tr>
<tr>
<td>1780.00</td>
<td>34.443</td>
<td>0.1001</td>
<td></td>
</tr>
<tr>
<td>2030.00</td>
<td>31.322</td>
<td>0.1101</td>
<td></td>
</tr>
<tr>
<td>2280.00</td>
<td>29.671</td>
<td>0.1203</td>
<td></td>
</tr>
<tr>
<td>2530.00</td>
<td>26.390</td>
<td>0.1307</td>
<td></td>
</tr>
<tr>
<td>2780.00</td>
<td>24.407</td>
<td>0.1413</td>
<td></td>
</tr>
<tr>
<td>3030.00</td>
<td>22.667</td>
<td>0.1521</td>
<td></td>
</tr>
<tr>
<td>3280.00</td>
<td>21.130</td>
<td>0.1632</td>
<td></td>
</tr>
<tr>
<td>3530.00</td>
<td>19.761</td>
<td>0.1745</td>
<td></td>
</tr>
<tr>
<td>3780.00</td>
<td>18.536</td>
<td>0.1860</td>
<td></td>
</tr>
<tr>
<td>4030.00</td>
<td>17.432</td>
<td>0.1978</td>
<td></td>
</tr>
<tr>
<td>4280.00</td>
<td>16.435</td>
<td>0.2098</td>
<td></td>
</tr>
<tr>
<td>4530.00</td>
<td>15.528</td>
<td>0.2221</td>
<td></td>
</tr>
<tr>
<td>4780.00</td>
<td>14.701</td>
<td>0.2346</td>
<td></td>
</tr>
</tbody>
</table>

---

126
TABLE V-11. Land Use Pollutant Accumulation Rates

<table>
<thead>
<tr>
<th>Land Use</th>
<th>Biochemical Oxygen Demand, lbs/acre/day (kg/hectare/day)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>HEC&lt;sup&gt;1&lt;/sup&gt;</td>
</tr>
<tr>
<td>Low Density Residential</td>
<td>0.04 (0.04)</td>
</tr>
<tr>
<td>Medium Density Residential</td>
<td>0.07 (0.08)</td>
</tr>
<tr>
<td>High Density Residential</td>
<td>0.13 (0.15)</td>
</tr>
<tr>
<td>Commercial</td>
<td>0.46 (0.52)</td>
</tr>
<tr>
<td>Industrial</td>
<td>0.39 (0.44)</td>
</tr>
<tr>
<td>Open Space and Rural</td>
<td>0.02 (0.02)</td>
</tr>
<tr>
<td>Pastures</td>
<td>3.10 (3.47)</td>
</tr>
<tr>
<td>Farming</td>
<td>0.02 (0.02)</td>
</tr>
<tr>
<td>Forests (Douglas Fir)</td>
<td>0.01 (0.01)</td>
</tr>
</tbody>
</table>

<sup>1</sup> Hydrologic Engineering Center (1977).

<sup>2</sup> Triangle J Council of Governments (1976), average values for Orange, Wake and Durham counties, North Carolina (in the Piedmont).

<sup>3</sup> Modifications based on examination of water quality data for Salem Creek, all other values as recommended by HEC.
TABLE V-12. Average Values of Key STO/TRT and Level III - RECEIVING Parameters and Hydraulic Variables

<table>
<thead>
<tr>
<th>Time Period Of Simulation</th>
<th>Velocity, ft/sec (m/sec)</th>
<th>Depth, ft (m)</th>
<th>Longitudinal Dispersion ft²/hour (m²/hour)</th>
<th>Deoxygenation Rate 1/hour (1/day)</th>
<th>Reaeration Rate 1/hour (1/day)</th>
</tr>
</thead>
<tbody>
<tr>
<td>July 16 to August 3, 1981</td>
<td>0.63</td>
<td>2.13</td>
<td>290,676</td>
<td>0.1238</td>
<td>0.1546*</td>
</tr>
<tr>
<td></td>
<td>(0.19)</td>
<td>(0.65)</td>
<td>(27,005)</td>
<td>(2.97)</td>
<td>(3.71)</td>
</tr>
<tr>
<td>November 4, 1980 to August 25, 1981</td>
<td>0.72</td>
<td>2.47</td>
<td>392,369</td>
<td>0.0891</td>
<td>0.1272*</td>
</tr>
<tr>
<td></td>
<td>(0.22)</td>
<td>(0.75)</td>
<td>(36,453)</td>
<td>(2.14)</td>
<td>(3.05)</td>
</tr>
</tbody>
</table>

*DEM Intensive Survey average value for 1976 was 3.07 per day (0.128 per hour) for Salem Creek.
DURING WET WEATHER PERIODS

Figure V-25. Comparison of Deterministic Steady-State Versus Transient Simulation, Salem Creek

Figure V-26. Comparison of Deterministic Steady-State Simulation Versus Statistical Prediction, Salem Creek
remarkably close, which indicate that the choice of the simple statistical model is a good one for preliminary planning purposes.

The highest level of analysis proposed involves deterministic continuous simulation of receiving water quality transients (Medina, 1982). Not only frequency, but also duration of water quality violations are predicted. With computer graphics, three-dimensional plots of water quality concentrations in time and space can be viewed. Cause and effect, BOD-DO concentration profiles are presented in Figures V-27 to V-30.

The frequency distribution of the duration of violations of a 5.0 mg/l stream DO standard is presented in Figure V-31 for Salem Creek, at x = 1.90 miles downstream from the Archie Elledge WWTP, for the entire period of simulation. For example, the figure reveals that there were 8 occurrences during which the standard was violated for 12 consecutive hours and one occurrence of a violation for 60 consecutive hours.

5.5 DISSOLVED OXYGEN CRITERIA FOR FISH

Although the methodology presented in this study and demonstrated in the preceding section has been applied to a fixed stream DO standard, it has been noted that cumulative frequency curves and frequency distributions of duration are easily adaptable to dynamic standards. In a comprehensive study of dissolved oxygen criteria for freshwater fish Warren, Doudoroff and Shumway (1973) noted that evidence had been accumulated to indicate that temperature acclimatization, necessary activity, feeding status of the fish, and seasonal changes in the fish all influence their oxygen requirements for survival. Experimental results indicated rapid recognition and avoidance by some fish (salmonid and centrarchid) of water with reduced oxygen concentrations well above those known to be lethal for these fish. They also noted, however, that the same oxygen concentrations are not necessarily avoided in the same way under more natural conditions. Largemouth bass tested at 25°C showed impairment of sustained swimming performance only when oxygen was reduced to levels below 5 or 6 mg/l. Growth rate increased slightly with increasing availability of oxygen, but not so much as food consumption rate, because respiration increased, primarily as a result of increased specific dynamic action. It was the ability of the fish to increase their rate of respiration with increase in oxygen availability that permitted consumption rate and, in consequence, growth rate to increase. Oxygen was acting as a limiting factor.

Toxic effects of chronic (30 day) and acute (24 hour) exposure of non-salmonid fish to various dissolved oxygen levels are summarized in Figure V-32. This type of information, coupled with frequency and duration predictions, provides a framework for assessing instream flow needs from a water quality viewpoint.
Figure V-27. BOD Concentration in Time and Space, July 16 - August 3, 1981, Salem Creek

Figure V-28. DO Concentration in Time and Space, July 16 - August 3, 1981, Salem Creek
BOD CONCENTRATION, MG/L
SIMULATION PERIOD: NOV. 15 - NOV. 29, 1980
SALEM CREEK - WINSTON-SALEM, N.C.

DO CONCENTRATION, MG/L
SIMULATION PERIOD: NOV. 15 - NOV. 29, 1980
SALEM CREEK - WINSTON-SALEM, N.C.

Figure V-29. BOD Concentration in Time and Space,
November 15 - November 29, 1980,
Salem Creek

Figure V-30. DO Concentration in Time and Space,
November 15 - November 29, 1980,
Salem Creek
### Figure V-31. Frequency Distribution of Duration of Violations of the Stream DO Standard, Salem Creek

<table>
<thead>
<tr>
<th>Consecutive Hours of D.O. Violation</th>
<th>Frequency</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>1</td>
</tr>
<tr>
<td>8</td>
<td>2</td>
</tr>
<tr>
<td>16</td>
<td>3</td>
</tr>
<tr>
<td>24</td>
<td>4</td>
</tr>
<tr>
<td>32</td>
<td>5</td>
</tr>
<tr>
<td>40</td>
<td>6</td>
</tr>
<tr>
<td>48</td>
<td>7</td>
</tr>
<tr>
<td>56</td>
<td>8</td>
</tr>
<tr>
<td>64</td>
<td>9</td>
</tr>
</tbody>
</table>

### Table: Toxic Effects Non-Salmonid Fishery

<table>
<thead>
<tr>
<th>Dissolved Oxygen (mg/L)</th>
<th>Chronic (30 Day Exposure)</th>
<th>Acute (24 Hour Exposure)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Embryo &amp; Other Larval Stage Life Stages</td>
<td>All Life Stages</td>
</tr>
<tr>
<td>10</td>
<td>No Production Impairment</td>
<td>Essentially Safe</td>
</tr>
<tr>
<td>9</td>
<td>Slight</td>
<td>Increasing Non-Lethal Stress</td>
</tr>
<tr>
<td>8</td>
<td>Moderate</td>
<td>Increasing Individual Mortality</td>
</tr>
<tr>
<td>7</td>
<td></td>
<td></td>
</tr>
<tr>
<td>6</td>
<td></td>
<td></td>
</tr>
<tr>
<td>5</td>
<td></td>
<td></td>
</tr>
<tr>
<td>4</td>
<td></td>
<td></td>
</tr>
<tr>
<td>3</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>Severe Stress</td>
<td></td>
</tr>
<tr>
<td>0</td>
<td>No Survival</td>
<td></td>
</tr>
</tbody>
</table>

**Figure V-32. Effects of Chronic and Acute Exposure of Non-salmonid Fish to Dissolved Oxygen Levels (modified after Driscoll, 1981)**
5.6 ADEQUACY OF DATA BASE FOR MODELING

Within budgetary and time constraints, there may be numerous sampling strategies that can be used to satisfy the objectives of the data collection agency and/or policy maker. If general trends or overall water quality appraisals at the regional level are desired, a widespread low-density network may suffice. Network planning and design should nevertheless emphasize collection of representative samples and transferability of the data to ungaged/unsampled sites. Important tradeoffs must be made between the choice of data types, number of catchments or basins to be gaged/sampled, the density of instrumentation, the frequency of gaging/sampling and the duration of gaging/sampling at each site. Inventories should be performed of river basins or sub-basins that would provide a representative sample of the hydrologic, geographic, topographic, and demographic characteristics of the region: for example, the Piedmont Province of North Carolina. Calibration and verification of continuous hydrologic and water quality simulation models will require large amounts of data in short time increments (high frequency) at relatively closely spaced intervals (high-density). The former is important to obtain frequency and duration of water quality fluctuations resulting in water quality violations or fish kills. The density depends on the size and number of tributaries and whether significant waste sources are located on these branches.

Although in a few cases model calibration, verification and routine application for a multiple land-use drainage basin may be accomplished with flow and water-quality data at a single point -- the resulting calibrated model parameters may certainly not be applicable to other basins with different distributions of land-use types even if the hydrologic inputs are identical. Water-quality samples are of little or no value if concurrent flow data are not available: for example, statistical analyses of data on concentrations are worthless if the values are not flow-weighted. Sites for the collection of receiving water quality should be at well-mixed flow reaches. Sampling may be required at different depths and across different vertical panels of the receiving water body -- much like velocity measurements are made to obtain a cross-sectionally averaged flow rate. Routine periodic sampling programs are not generally designed to assess specific problems (e.g., instream flow needs). Thus, data generated for general purposes are usually inadequate for defining cause/effect relationships that control most receiving water quality changes. Short-term intensive studies (such as the DEM intensive survey report, Salem and Muddy Creek, 1976) may be required during each season or critical period with three or four samples per day collected at each site, and more frequent sampling during storm flow conditions. The frequency should be high enough to display both diel variations (of or pertaining to 24-hour day) and seasonal variations. The time of measurement (in situ) or collection (for laboratory analysis) must be
recorded accurately: absolutely essential for modeling transient water quality.

The principal investigator is certainly not advocating the collection of data for the calibration and verification of mathematical models as an end in itself. However, to realistically assess all of the complex variables that influence water quality in a riverine environment—which in turn influence the suitability of the water body for its intended use—the application of these planning tools is inevitable. Even with an unlimited budget for collection of flow and water quality data, only a historical perspective is achieved. Models which have been properly adjusted to local conditions may be used to predict future water quality by selectively varying input such as: projected land use, synthetic rainfall, projected point source flows and pollutant concentrations (municipal, industrial, agricultural), projected changes in upstream flows due to diversions, etc.

For this particular study, a thorough search for available data (evaporation, land use, topography, rainfall, streamflow, fish species, water quality, population density, stage-discharge relationships, stream depths and velocities, deoxygenation and reaeration coefficients, etc.) was conducted by: computerized and manual literature review; computerized access to state and national stream flow and water quality storage and retrieval services, as well as personal communication with agency personnel; and interviews with local officials, state agency investigators and planning commissions. Evaporation stations were found to be farther apart than desired, with major time gaps existing in their records. Nevertheless, through careful weighting and interpolating a successful calibration for the hydrologic simulation was achieved. Although rainfall data were excellent for both general synoptic analysis and for detailed continuous hydrologic simulation purposes, recording of very short-time increment rainfall within the study drainage basin was a result of a temporary intensive study and will likely not be continued ad infinitum. The most complete concurrent records of all the variables required for long-term continuous computer simulation in the entire Yadkin River basin in North Carolina, for which the site also met the criteria of a riverine environment subjected to a wide variety of land uses, were found in the Salem Creek drainage basin.

The simplified statistical models applied in this study offer a significant advantage over the deterministic models: data requirements are much more modest. The integrated, hierarchical approach proposed for instream flow strategies allows the user to progress towards more sophisticated levels of analysis gradually. Advances in microelectronics portend the development of yet more powerful microcomputers, increasing the availability of these tools to a greater number of users as costs are reduced: making modeling packages more appealing for many applications.
REFERENCES


Buzun, Jennifer, Technical Services, Division of Environmental Management, North Carolina Department of Natural Resources and Community Development, personal communication, 1982.


Land Resources Information Service, Department of Natural Resources and Community Development, Raleigh, North Carolina, 1981.


Maya, Silvia M., City-County Planning Board, City Hall, Winston-Salem, North Carolina, personal communication, 1982.


