CLEMSON

Report No. 120

WRRI

METHODOLOGY FOR ESTABLISHING COMPREHENSIVE INSTREAM REQUIREMENTS AND PREDICTING FLOW DEFICIENCIES

by

Soronadi Nnaji
Department of Civil Engineering
FAMU/FSU College of Engineering
Tallahassee, Florida 32316-2175

and

Earl J. Hayter
Department of Civil Engineering
Clemson University. Water resources research institute
Clemson, S.C. 29634-0911

September 1985

TECHNICAL COMPLETION REPORT

G-868-03
G-932-03

METHODOLOGY FOR ESTABLISHING COMPREHENSIVE INSTREAM REQUIREMENTS AND PREDICTING FLOW DEFICIENCIES

by

Soronadi Nnaji
Department of Civil Engineering
FAMU/FSU College of Engineering
Tallahassee, Florida 32316-2175

and

Earl J. Hayter
Department of Civil Engineering
Clemson University
Clemson, South Carolina 29634-0911

S.C. Water Resources Research Institute
Clemson University
Clemson, South Carolina 29634-2900

September 1985

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

Project Agreement Numbers

14-08-001-G-868-03
14-08-001-G-932-03

Period of Investigation

August 1, 1983 - September 30, 1985

ABSTRACT

The State of South Carolina has experienced a sustained increase in offstream demand over the past few years. This increased demand has resulted in instream demand for waterbased recreation, fish and wildlife propagation. The State Water Resources Commission has identified stream segments for which minimum flow levels need to be established as a prelude to preparing stream standards. The goal of this project was to develop a methodology that can be applied for establishing such standards. The objectives were to develop and illustrate a method for establishing instream requirements and for statistically predicting flow deficiency patterns with respect to these requirements for selected stream segments.

The basic approach used was to generate sequences of stream attributes which may include stream flow, stream quality and stream biota parameters. Crossing level analysis is performed for each sequence. The threshold for each attribute is the candidate instream requirement. The requirement may correspond to a stream standard for the attribute. Water withdrawal from the stream segment is used as the threshold for stream flow; the $7Q_{10}$ being the minimum threshold value for flow.

Measures of effectiveness of the requirement are computed from derived variables, such as the negative and positive run sums, that result from the crossing level analysis. The measures used are the stream reliability, resiliency and vulnerability. Other measures such as costs and benefits forgone may be included in the multiattribute - multimeasure decision analysis which is performed to select the optimal instream requirement vector.

The product includes two computer programs. The first program is an extension of the simplified stormwater management model developed by Lager et. al. (1976) to simulate, for given design hyetographs, the impact of detention storage overflows on the stream quality at down-stream locations. The second program automates the decision analysis for the selection of the instream requirement. The product will serve as a decision tool for construction of irrigation and municipal water withdrawal schedules and for maintaining instream requirements.

ACKNOWLEDGMENTS

The authors wish to express appreciation for the use of the computing facilities of Clemson University, Clemson, South Carolina and those of the FAMU/FSU College of Engineering, Florida Agricultural and Mechanical University, Tallahassee, Florida.

TABLE OF CONTENTS

TABLE OF CONTENTS (con't)

LIST OF FIGURES

LIST OF TABLES

I INTRODUCTION

1.1 PROBLEM STATEMENT

The State of South Carolina has experienced a sustained increase in offstream surface water demand over the past few years due to population growth, expanded use of agricultural irrigation systems and new industry development. This increased demand has adversely impacted flow dependent instream uses such as navigation, fish and wildlife resources, waste water assimilation, water quality and aesthetics.

Currently, South Carolina lacks the comprehensive statutory means to adequately protect instream uses. Partial protection may be possible through State and Federal legislation designed for other purposes such as water quality certification, scenic river designation, navigation, interbasin transfer, and drought response planning. To better assess the instream flow situation in South Carolina, the State Legislature in 1983 directed the Water Resources Commission 'to identify and list the streams for which minimum flow levels need to be established and prepare proposed streamflow standards'. During Phase I of that study 503 stream segments were evaluated for potential instream use problems based on natural and man-induced flow impacts and significance of use. Fifteen primary segments were identified (Figure 1).

Two pieces of legislation, The South Carolina Drought Response Act and the Interbasin Transfer Act, both of which were enacted in 1985, include direct reference to the

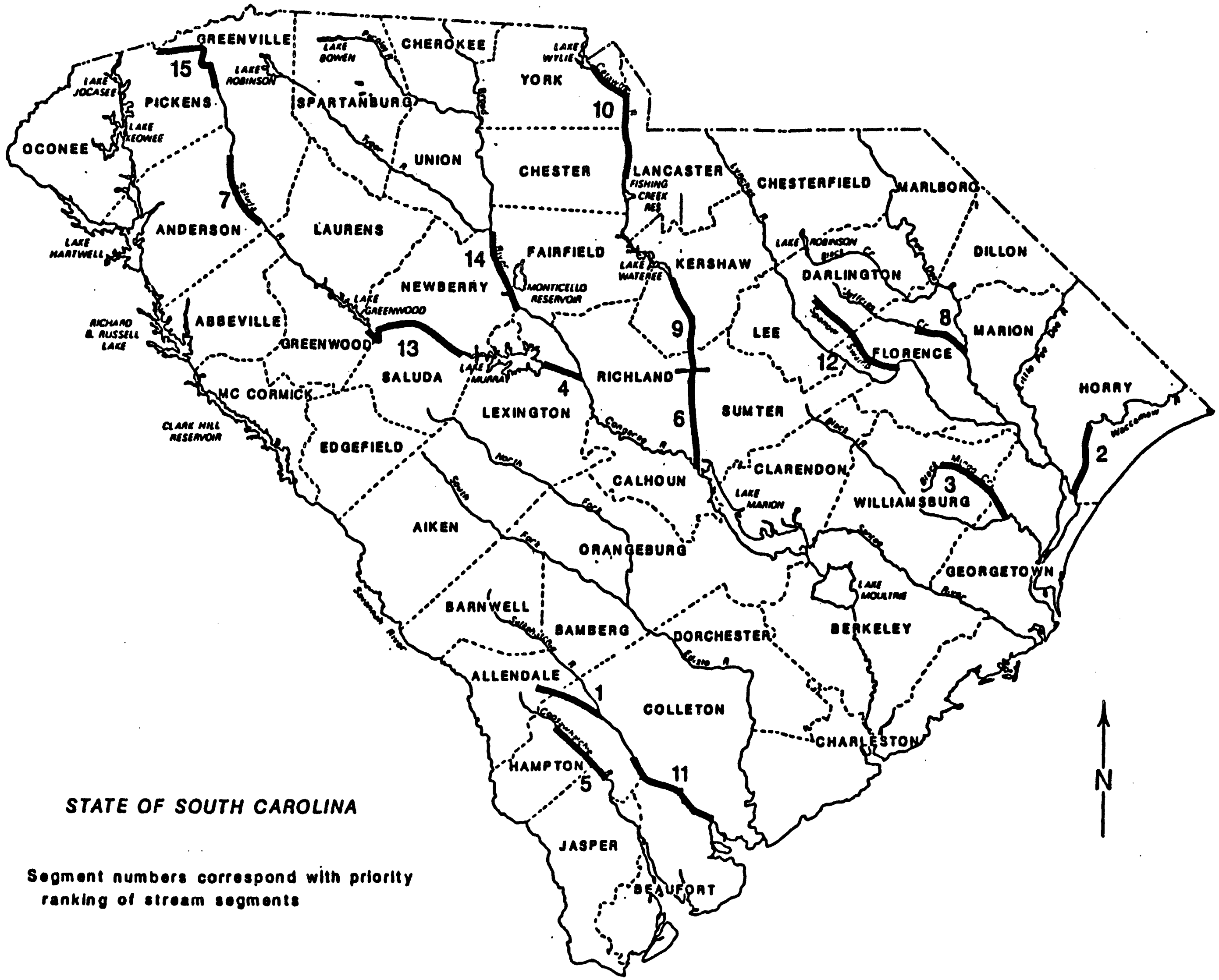

Figure 1. Location of Selected Problem Stream Segments.

protection of instream uses. The implementation of these statutes will require the analysis of offstream and instream use impacts under various flow and stream quality conditions.

1.2 OBJECTIVE OF THE STUDY

The objective of this study was to develop decision making tools for determining optimum stream discharge and quality requirements for multiple instream uses. The simultaneous consideration of multiple uses and measures for evaluating each instream requirement vector was an important consideration.

The decision problem of establishing an optimum instream requirement vector was addressed by examining two methods of Multiattribute - Multimeasure Decision Making --- the Simple Additive Weighting method and the Goal Programming method.

1.3 ORGANIZATION OF THE REPORT

The concept of instream requirement as viewed in this study is described in the first section of Chapter 2. Section 2.2 reviews the different approaches that have been used to obtain optimum instream requirements. The review concentrates on approaches that seek to model the urban catchment and the impact of storm runoff quality on the stream regime downstream of the outfall.

Two classes of multiobjective models, Goal Programming and Value-Based models, are reviewed in Section 2.3. Section

2.4 provides an application of both classes of models to a practical water supply management problem.

The methodology of this study is presented in Chapter 3. The lag-one Markov model for generating attribute sequences is described in Section 3.1. In Section 3.2 a modification of the Simplified Storm Water Management Model by Lager et. al.(1976) is described.

The multiattribute-multimeasure decision model is described in Section 3.3. Expressions for the measures of effectiveness of the candidate Instream Requirement Vector (IRV) are developed in Subsection 3.3.1. The expressions for the decision analysis are described in Section 3.3.3 for both the Goal Programming and Value-Based formulations.

The methodology is applied in Chapter 4. The information needs and available data are reviewed in Section 4.1. In Section 4.2, the statistical model is used to generate sequences of stream attributes and the Value-Based Approach is used to obtain the Instream Requirement Vector.

II LITERATURE REVIEW

2.1 DEFINITION OF INSTREAM REQUIREMENT

The quantity of flow that is necessary to sustain a desired level of activity (use) within a stream segment is referred to as the instream flow requirement. Similarly, the stream quality, with respect to a specified attribute, necessary to sustain a desired level of activity within the segment is referred to as the instream quality requirement. For our purpose the desired level of activity is simply referred to as the instream requirement whether the attribute of interest is quantity, quality or both quantity and quality related.

Instream requirements will vary in time, space and among uses, some of which are conflicting. For example, the need to use the stream as a waste receptacle versus the use of the stream as a medium for fish propagation. There exists a number of requirements for the various uses with respect to a given attribute (Flow, DO, BOD, Fish counts, etc.). Consider instream uses that are dependent upon large quantities of water. If the requirement for the most demanding use is selected as the instream flow requirement, the requirements for all other uses are satisfied. However, this may cause a significant benefit loss due to the unavailability of some of this water for offstream use. Thus, there exists a need for a tradeoff between instream and offstream uses which may be measured in terms of costs or some other measure of effectiveness.

Most of the previous instream requirement studies have examined such tradeoffs for only one stream attribute, usually streamflow. These studies were also usually performed using one measure of effectiveness. This study considers the case of multiple attributes and multiple measures simultaneously. The literature reviewed below categorizes possible approaches used to obtain an optimum instream requirement.

2.2 METHODS FOR DETERMINING INSTREAM REQUIREMENTS

2.2.1 Market Approaches

Daubert, Young and Gray (1979) used techniques developed for estimating the value of public goods to sample recreationists using a Colorado mountain stream to determine their willingness to pay for alternative rates of flow. Regression techniques were applied and the results used to estimate the marginal value of instream flows for each month of the recreation season.

Walsh et. al. (1980) performed analyses similar to those of Daubert, Young and Gray (1979). Respondents to interviews at nine river sites reported their willingness to pay, contingent on changes in congestion and water level in the stream. Walsh (1980) extended this effort by using the above criteria of congestion and water level to estimate the recreational value of water in reservoirs compared to instream flow.

Narayanan et. al. (1983) noted that one of the difficulties of integrating instream uses into the appropriation system of water law is the fact that instream flow uses are considered more environmental than economic in character. They followed the approach of reconciling benefits and costs of instream flow uses and developed theoretical aspects as well as a practical procedure for instream flow benefit estimation. A stochastic linear programming model was used to estimate the expected costs of alternative methods to maintain instream flows. A direct conflict between offstream agricultural use and the maintenance of instream flows was assumed.

2.2.2 Biota Based Approaches

The Division of Biological Services of the Fish and Wildlife Service, U.S. Department of the Interior, have conducted extensive work in instream flow analysis and management under its Cooperative Instream Flow Program. Two basic groups of reports have emerged from these studies - the Instream Flow Information Paper Series and the Opportunities to Protect Instream Flows. The former reports are technical in nature and center around the development and implementation of the "Instream Flow Incremental Methodology" (IFIM) for the assessment of riverine habitats and the evaluation of remedial measures to counter man's actions that lead to stream habitat degradation. Specifically, the report by Bovee (1982) demonstrates the application of the methodology. The latter group of reports provide basic surveys of state prerogatives and programs that may be used to protect instream uses of water. Sale, Grill and Herricks (1982) have proposed a mathematical programming methodology to examine the relationship between biological instream flow needs and more traditional water project objectives such as water yield, flood control, reservoir recreation or economic efficiency. This optimization approach combined the linear decision rule modeling technique with an objective function representing the value of reservoir releases to downstream fisheries.

2.2.3 Hydrologic Simulation Approaches

Medina (1983) developed a framework to address the impact of water quality fluctuations in determining instream flow strategies. Continuous hydrologic and water quality simulation models were used to derive the frequency and duration of water quality standards violations. The method was applied to two streams in the Yadkin-Pee Dee River Basin, North Carolina.

Wallace et. al. (1980) modified an existing watershed simulation model in order to obtain time series of flow in six ungaged Georgia streams. They found that simulated low flows were considerably more accurate for some watersheds than for others. The authors also used available data on many continuous and partial record gaging stations. Zones with similar 7-day average low flows were outlined for different return periods. They also examined the use of the correlation between precipitation and low flow as a method for predicting low flow. Values of the 7-day, 10-year flow developed by the authors may be used as a minimum instream flow requirement for the streams for which they were developed.

Singh and Stall (1974) determined the 7-day, 10-year low flow every 3 to 4 miles along selected streams in Illinois. They used data from gaging stations, regional low flow vs drainage area curves, effluent vs population relationships, and took into account soil properties, groundwater hydrology and man-made structures.

Bloxham (1981) has applied regionalization methods to define the low-flow characteristics in the Piedmont and the Lower Coastal Plain physiological provinces of South Carolina. The regression equations with standard errors (SE) of estimate developed by Bloxham are:

$$7Q_2 = 0.17\ A^{0.94}E^{-0.03}D_{95}^{\ 0.89} \qquad SE = 27\% \qquad (2.1)$$

$$7Q_{10} = 0.16\ A^{0.87}E^{0.15}D_{95}^{\ 1.32} \qquad SE = 34\% \qquad (2.2)$$

where $7Q_T$ is the 7-day discharge of T-year recurrence interval in cubic feet per second, A is the contributing area in square miles, E is the mean basin elevation in feet above sea level, and D_{95} is the flow rate at 95% duration in inches per year. The variable D_{95} reflects aquifer yield under definitive base flow conditions. The values computed for $7Q_{10}$ can be used as the lower bounds of instream flows. For a given level of instream flow above this minimum, many different instream uses can take place.

In theory, the error of the estimate of the $7Q_T$ discharge decreases with sample size. In practice, regulation and diversion, among other activities, are continually changing the quality and quantity regime of the stream segments. The result is a continued need to revise these estimates to reflect the non-stationarity of the stream environment.

2.3 CLASSES OF MULTIOBJECTIVE MODELS

The Value-Based (VB) and the Goal Programming (GP) classes of models are of interest in this study and are briefly reviewed below with a numerical example.

2.3.1 Goal Programming Models

GP models are designed to minimize the set of deviations from prespecified multiple goals. These goals are considered simultaneously and weighted according to their relative importance to the Decision Maker (DM). Preemptive goal programming models, in contrast, first determine the alternatives that minimize the deviation of the most important objective from its corresponding goal value. From this subset of alternatives, a further subset of alternatives that minimize the deviation of the next most important object from its goal are selected. This sequential process continues until all objectives are considered.

Goal Programming Formulation: The two subclasses of models differ only in their handling of the objective function. The formulation of goals and constraints are the same. These are written as:

$$\Sigma c_{ik} x_k + g_i - h_i = b_i \qquad i=1,m \qquad (2.3)$$

$$x_k \geq 0 \quad , \; g_i \geq 0 \quad , \; h_i \; \geq 0$$

where x_k is the k-th decision variable, g_i is the negative deviation, h_i is the positive deviation from the goal value ,b_i , of the i-th goal. The variable c_{ik} is the technological coefficient of the i-th goal and k-th decision variable. If b_i is a fixed upper limit then g_i is a slack variable and h_i is omitted from the constraint, which is of the less than or equal to type. Conversely, if b_i is a fixed lower limit, h_i is a surplus variable and g_i is omitted from the constraint, which is of the greater than or equal to type. If b_i is a goal (i.e. not fixed in value) then either g_i or h_i or both appear in the objective function.

The objective function is given, for the nonpreemptive goal programming formulation, as:

$$Z \;=\; \min \;\; \Sigma \; [w_i \;\{f_i(g_i,h_i)\} \;]^a \qquad (2.4)$$

where f_i is a function, usually linear, of the deviational variables for the i-th objective and w_i is the corresponding importance weight of the objective. The value of the exponent, a, is typically 1 or 2. For the preemptive goal programming formulation the objective function, Z, is:

$$Z \;=\; \min_{k=1,K} \; f_i(g_i,h_i,k_i) \qquad (2.5)$$

For linear f_i one may write in general:

$$f_i = [w_i g_i + (1-w_i)h_i] \qquad (2.6)$$

where k_i=1,K is the rank of the i-th variable.

Since this class of models considers deviations from goals, they are particularly suited to situations where goals are in conflict which occur routinely in stream water use. While conflicting goals cannot be achieved simultaneously, deviations from the goals can be so minimized. Goal programming is an offshoot of the Linear programming (LP) technique for optimum resource allocation. It therefore benefits from the many extensions, such as post optimality analysis, which has made the LP technique a very powerful tool.

The objective function, constraints and goal relationships must be linear in goal programming. However, stream resource allocation scenarios leading to non-linear relationships among the above items may be expected to occur on occasions. Goal programming cannot be readily adapted to solve non-linear resource allocation problems.

2.3.2 Value-Based Models

This class of models attempts to solve multiobjective problems by calculating some form of expected value of a criterion function for each alternative. The alternative with the minimum (maximum) expected value is selected. Membership in this class include Utility models and Cost-Effectiveness models. Most applications of these models assume independence between objectives so that multiple goals may be attained simultaneously. Instream management may have interdependent goals.

Formulation: The structure of the models is best depicted in the decision matrix given in Table 1 below. As shown in the last column of this table, the value of each alternative is obtained by summing the weighted criterion function value for each objective. In the context of this study, the criterion function may be given in terms of the deviation of the attribute value from the corresponding instream requirement. Hence f may be given by:

$$f = |y - x_{ij}|^{\alpha} \tag{2.7}$$

where y is the requirement, x_{ij} is the value of the attribute and α is an exponent which reflects the relative importance of large and small deviations. All deviations are weighted equally when $\alpha=1$, whereas for $0<\alpha<1$ small deviations weighted more than large deviations. The reverse is true for $1<\alpha\leq\infty$.

The Utility models use utility functions as the criterion function. These functions reflect the decision makers attitude towards the risk associated with the corresponding objective. It is however very difficult to accurately assess the Decision Maker's utility function.

Table 1. Multiattribute Single-Measure Decision Matrix

Attribute / Alternative	A_1	A_3	A_j	A_n	Figure of Merit
y_1	p_{11}	p_{31}	p_{j1}	p_{n1}	W_1
.	.	.	.	.	.
.	.	.	.	.	.
.	.	.	.	.	.
y_k	p_{1k}	p_{3k}	p_{jk}	p_{nk}	W_k
.	.	.	.	.	.
.	.	.	.	.	.
y_K	p_{1K}	p_{3K}	p_{jk}	p_{nK}	W_K

y_k = The k-th alternative
A_j = The j-th attribute
w_j = The importance weight of the j-th attribute
p_{jk} = The probability of meeting the goal with respect to the j-th attribute through alternative k
W_k = The figure of merit of alternative k
= $\sum_j p_{jk} w_j$

2.4 MULTIOBJECTIVE MODELING - ILLUSTRATIVE EXAMPLE

2.4.1 The Problem

A water supply problem in San Angelo, Texas (Erskine and Shih, 1972) was proposed to be solved by either of three alternatives:

A_1 : Continuation of present water management policies and complete reliance on rainfall;

A_2 : Continuation of present water management policies and development of adequate groundwater resources in nearby McCulloch county;

A_3 : Revision of current water management policies to include rationing and rate adjustment programs, the development of a short-range water supply to smooth rainfall fluctuations, and the initiation of waste water reuse programs.

The alternatives were evaluated based on the following objectives:

O_1 : Meeting future water demand;

O_2 : Improving water quality to meet minimum health requirement;

O_3 : Minimizing annual costs;

O_4 : Increasing recreational benefits such as water sports and lawn improvement;

O_5 : Social acceptance, considering social preferences based on factors other than the above.

Table 2 below summarizes the data for the problem. The importance weights are normalized values based on a group of expert opinions. The entry for each alternative is

Table 2. Decision Matrix for Water Management Example

Objectives	Meet Future Demand	Water Quality	Annual Cost	Recr. Benefit	Social Accept.
Relative Weights	44.9	23.8	15.8	11.1	4.4
Alternatives					
A1	0.7865	0.6	1.0	0.0	1.0
A2	1.0	0.9	0.0	0.91	1.0
A3	0.9995	0.8	0.472	0.77	0.0

essentially the probability that it will meet or exceed the objective. The goal column contains the goal value which has been normalized to be equal to 1 for each objective. Depending on the objective, this value may be the upper limit, the lower limit or an exact requirement and set equal to 1. For example, in cost minimization this value is the upper limit while it is the lower limit for objectives such as improving water quality.

2.4.2 Goal Programming Formulation

Each alternative has a separate formulation, though the objective function remains the same for all of them. Let x_i represent the acceptable level of the i-th goal. Since the goal is 1 for all of the goals, $x_1 \leq 1$. Recall that the entry in Table 2 for each row is the probability, p_i, of meeting or exceeding the objective. Therefore, the probable level of achievement is $p_i x_i$. The corresponding deviation, d_i, from the goal is

$$\begin{aligned} d_i &= h_i - g_i \\ &= p_i x_i - 1 \end{aligned} \qquad (2.8)$$

i.e. $p_i x_i - (h_i - g_i) = 1$ for all x, $d \geq 0$. For this example we take $x_i = 1$ for all i. The formulation for each alternative may now be written:

Alternative A_1

$$\begin{aligned} 0.7865\ x_1 - (h_1 - g_1) &= 1 \\ 0.6\ x_2 - (h_2 - g_2) &= 1 \\ 1.0\ x_3 - (h_3 - g_3) &= 1 \\ 0.0\ x_4 - (h_4 - g_4) &= 1 \\ 1.0\ x_5 - (h_5 - g_5) &= 1 \end{aligned} \qquad (2.9)$$

Alternative A_2

$$\begin{aligned} 1.0\ x_1 - (h_1 - g_1) &= 1 \\ 0.9\ x_2 - (h_2 - g_2) &= 1 \\ 0.0\ x_3 - (h_3 - g_3) &= 1 \qquad (2.10) \\ 0.91\ x_4 - (h_4 - g_4) &= 1 \\ 1.00\ x_5 - (h_5 - g_5) &= 1 \end{aligned}$$

Alternative A_3

$$\begin{aligned} 0.9995\ x_1 - (h_1 - g_1) &= 1 \\ 0.8\ x_2 - (h_2 - g_2) &= 1 \\ 0.472\ x_3 - (h_3 - g_3) &= 1 \qquad (2.11) \\ 0.77\ x_4 - (h_4 - g_4) &= 1 \\ 0.0\ x_5 - (h_5 - g_5) &= 1 \end{aligned}$$

The objective function for all the alternatives is

$$\min Z = \Sigma\{w_i(\alpha_i h_i + \alpha_i g_i)\}$$

where w_i = importance weight for i-th goal

α_i^+ = penalty for positive deviation from goal

α_i^- = penalty for negative deviation from goal

For objectives 0_1, 0_2, 0_4, 0_5 there is no penalty associated with positive deviations so that $\alpha_i^+ = 0$ for i = 1,2,4,5. For cost minimization (0_5) a positive deviation is not desirable so that $\alpha_3^+ = 0$. Conversely $\alpha_i^- = 0$ for i = 1, 2, 4, 5, and $\alpha_3^- = 0$. We assume that α_i^+ and α_i^- are equal to 1 for cases when they are not equal to 0. Thus the objective function is

$$\min Z = 44.9g_1 + 23.8g_2 + 15.8h_3 + 11.1g_4 + 44g_5 \quad (2.12)$$

The above three problems were solved using a commercial LP computer package. The solutions are:

Objective function value for $A_1 = 30.21$

$A_2 = 19.18$

$A_3 = 20.08$

The solutions, following the utility approach, are obtained by computing the overall Relative Utility, U_k, for the kth alternative,

$$U_k = w_i p_{ik} \quad (2.13)$$

so that $U_1 = 69.78$, $U_2 = 80.82$ and $U_3 = 79.92$. This is the same result as that obtained using the GP formulation. Note that the Utility approach seeks to maximize the utility of the alternative to the decision maker while the GP approach minimizes the deviations from the goal.

The GP approach gives more information than the Relative Utility Ranking method. Looking specifically at A_2, objectives 1 and 5 are met (since h_1, g_1, h_5, and g_5 are zero), but the A_2 option falls short of objectives 2, 3 and 4 ($g_2 = 0.1$, $g_3 = 1.0$ and $g_4 = 0.09$). Objective 3 is not met at all (since A_2 had the highest annual cost and therefore missed the goal of minimizing the annual costs by the most).

The LP program also allows the user to perform a sensitivity analysis on the coefficients and the right hand side values. This allows the analyst to investigate the importance of changes in the weights assigned to each objective (the objective function coefficients) and changes in the numerical goal values. In this particular example, the goals were all converted to a uniform scale of 1. This may not always be necessary. None of the coefficients in this example is sensitive to small changes in value. It is noted that the two approaches considered above, though multiobjective, use single measure (criterion) to choose between the alternatives. In Chapter 3 both multiple objectives and multiple measures are considered.

III METHODOLOGY

The methodology described in this section may be considered to be generic. The goal is to develop a procedure that can be applied to groups of stream segments that are under similar risks. The analyst can consider many alternative instream requirement vectors (IRV) as well as many stream attributes (quality and quantity parameters) and measures for evaluating the parameters. The number of attributes, alternatives and measures is limited by the available storage of the computing device and the speed of the processor(s).

Figure 2 shows the flow diagram of the methods for obtaining the optimum IRV. The sequence of attributes may be generated either from a statistical model or from an event-based rainfall-runoff - quality simulation model. The input data sets include stream quality (physical, chemical and biological) data collected by the South Carolina State Department of Health and Environmental Control (DHEC).

3.1 THE STATISTICAL METHOD

3.1.1 Preamble

Hydrologic records are usually not long enough to define the behavior of the variable of interest. It has been common practice to apply schemes which preserve the characteristics of the observed record in order to extend the data base to cover the design recurrence interval. Worse situa-

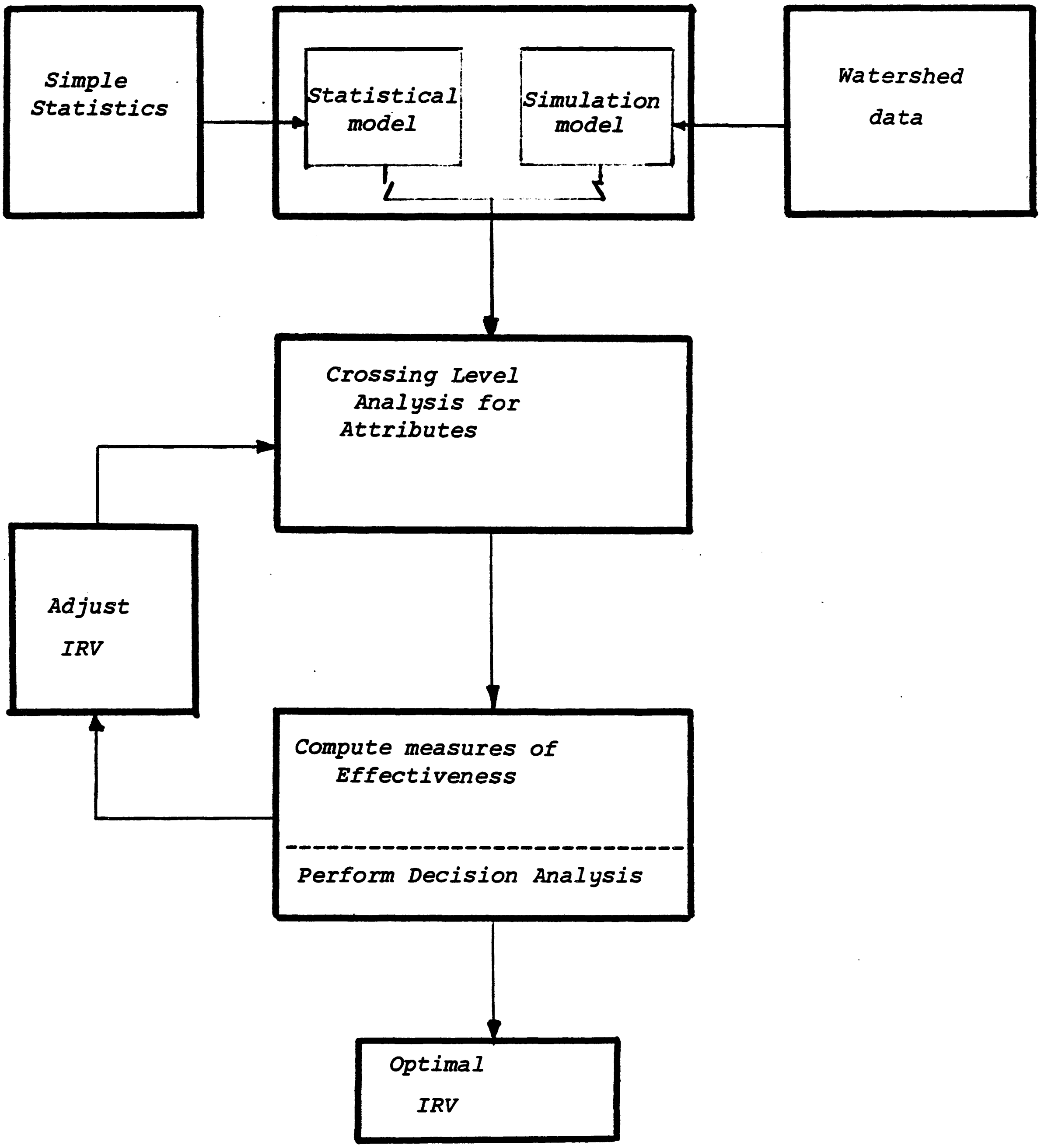

Figure 2. Flow Diagram of Methodology

tions occur when there are no partial records to base any scheme. The approach then is to apply regionalization and information transfer techniques to fill the data void.

The South Carolina Water Resources Commission (de Kozlowski, 1985) has evaluated 503 stream segments in South Carolina and ranked these stream segments based on the potential for instream use problems. One of these stream segments will be used in Section 4.2 to implement the methodology that is described in the next three sections.

3.1.2 <u>Generation of Attribute Sequences</u>

The work of Fiering and Jackson (1971) is the standard reference for the application of different generating schemes. The scheme applied in any study depends on the parent distribution of the data for the attribute of interest. For this, the available record must be analyzed to compute the sample statistics - mean, variance, skew and the lag one serial correlation coefficient. Based on the statistics and possibly the shape of the histogram a choice is made of a plausible distribution function for the attribute.

The first order Markov model is typically written as

$$x_{i+1} = x' + r_1(x_i - x') + (1-r_1)^{1/2}\, s_x . \varepsilon_{i+1} \qquad (3.1)$$

where x', s_x and r_1 are, respectively, the sample mean, standard deviation and lag-one correlation coefficient. The variable ε_{i+1} is the noise term of a parent distribution and is variously assumed to be Normal, Lognormal or Gamma distributed.

The above Markov model will be used to generate the sequences of attributes separately, in effect, assuming independence between them. A more rigorous approach would require the use of multivariate methods that consider the cross-correlation between the attributes. The generated sequences are used in the Crossing Level Analysis.

3.2 STORM WATER QUANTITY AND QUALITY MODELING

3.2.1 Introduction

The urbanization process results in an increase in runoff volumes and peak discharge rates as well as a deterioration in runoff quality. It also causes a decrease in the natural storage of the catchment due to the increase in the percentage of the area that is covered by impervious surfaces. Flow lengths are also decreased.

The major computer programs that have been developed to model stormwater quantity and quality dynamics have been briefly reviewed in Section 2.2.3. Some of these models are expensive to set up and complex to use. Others are very simplified models developed to reduce data collection and reduction costs at the expense of accuracy.

The model described in this section and identified in the flow diagram of Figure 2 is a modification of the Simplified Storm Water Management Model (SSWMM) developed by Lager et. al. (1976). The major extensions include a routine that constructs the joint empirical distribution of the duration and magnitude of storm events from a long sequence of hourly rainfall data. Given a design return period, a second routine inverts the above joint distribution to obtain the corresponding duration and rainfall peak intensity. It assumes one of two standardized storm patterns to obtain the design hyetograph which in turn is routed through the catchment to yield the design hydrograph. The hydrograph is then routed through the detention storage; overflows are discharged into a receiving stream. A third routine assumes a constant upstream flow and pollution content and input pollutograph (from the catchment) to predict the impact of overflows on the stream quality at downstream locations.

Methods presently used for determining the design peak flow of a certain return period assume that the watershed response to a certain storm event is linear, i.e. the return period of the design rainfall is equal to that of the flow calculated from it. A more accurate value of the design flow can be obtained if the linear response assumption is not used. The third program accomplishes this by determining the design peak flow from the statistical analysis of the peak flows resulting from different storm events, instead of determining the design peak flow of a certain return period from the design storm of the same return period.

The above programs are described below. The required input data and resulting output from the programs are given in Appendix A.

3.2.2 Storm Program

This program combines hourly rainfall data to obtain storm events of certain magnitudes and durations. Rainfall starts must be at least six hours apart to be considered separate events. Hourly rainfall data over a long period of time are essential for the definition and statistical analysis of storm events. The needed hourly rainfall data are most readily available from the National Weather Service (NWS) Climatic Data publications for the locality of interest. These data have to be correlated with local data if the closest National Weather Service gage is not close to the area being studied.

Initially, this program reads hourly rainfall data one day at a time, checks the occurrence of rainfall and the time of occurrence of the last rainfall. If it has just started raining and if the interval of time since the last recorded rainfall is greater than six hours, a new rainfall event is assumed to start and the magnitude of the previous event as well as its duration are stored. This process is continued until all the data are read.

After identifying all the storm events and determining their magnitudes and durations, the program determines the cumulative distribution functions and the joint distribution function (contingency type distribution function) of these magnitudes and durations. The magnitude and duration of the design storm for a given return period is then determined using the calculated joint cumulative distribution function.

3.2.3 Watershed-River Response Program

The watershed response portion of the program determines the runoff hydrograph of the watershed being analyzed. First, the main channels are identified. The runoff hydrograph from a subarea is calculated by the runoff subroutine at the point where this subarea discharges into one of the main channels.

The process is to calculate the runoff hydrographs from all the subareas, routing them to desired points in the watershed, and convoluting so as to obtain the final watershed runoff hydrograph. This is implemented using several subroutines whose functions are explained briefly below.

1- Event Distribution Subroutine: In this subroutine, the storm volume determined in the rainfall analysis program is distributed over the duration according to the Soil Conservation Service (SCS) type II dimensionless distribution. The storm volume and duration may be :

(a) the design storm volume and duration;

(b) the observed sequences of storm volume and duration;

(c) sampled from the joint distribution of storm volume and duration.

Case (c) is appropriate when the record is short and needs to be extended to cover the design storm return period. It may also be used when the project site is ungaged and data are transferred from other locations.

2- Subarea Runoff Calculation Subroutine: This subroutine calculates the runoff hydrograph from a subarea at the point where its runoff discharges to one of the main channels by applying rainfall-excess increments to the synthetic unit hydrograph of that subarea. These rainfall excess increments are computed from the rainfall increments by using a modification of the SCS method (rainfall excess increments being the rainfall increments available for runoff).

The subarea being analyzed is divided into as many sections as desired. The curve numbers for these sections are determined from tables given in SCS manuals.

3- Add Subroutine: This subroutine combines two hydrographs.

4- Route Subroutine: The runoff hydrograph at a point in the watershed is routed through a channel to another point using this subroutine. The routing algorithm is based on the Muskingum method.

The second part of the Watershed-River Response program determines the variation in the receiving water quality with time at different points downstream from the point of discharge of the watershed runoff to the stream using the river response subroutine, which is described below.

5- River Response Subroutine: This subroutine is based on a program written by Medina (1983) in which the dissolved oxygen deficit (DO) and the biochemical oxygen demand (BOD) are calculated at different points downstream from the point of discharge of the watershed runoff and at different points in time. The equations used by Medina were for a continuous input into the receiving stream. Note that the three input types listed above are event based. Sample input and output data from the Watershed-River response program for case (a) are given in Appendix A. The output for case (b) will be sequences of the flow hydrographs, DO pollutographs and BOD pollutographs on which a Crossing Level Analysis, as described in Section 3.1, may be performed. The hydrographs and pollutographs in case (c) will not be time sequences. This case is considered further in Subsection 3.3.1.

3.2.4 The Design Peak Flow Program

The sequence or replications of hydrographs, depending on the input into the Watershed-River response program, are further analyzed in the Design Peak Flow program. Additional input are the minimum rainfall volume above which peak flow is to be calculated, and the return period of the design peak flow to be calculated.

The program identifies the peak of each input hydrograph and obtains the empirical cumulative distribution function (CDF) as well as the sample statistics of the peaks. It inverts the CDF for the given return period to obtain the corresponding design peak discharge as output. An example of the required input and output data for the Peak flow program are given in Appendix A.

3.3 MULTI-ATTRIBUTE MULTI-CRITERIA DECISION ANALYSIS

3.3.1 Expressions for Measures of Effectiveness

Consider the time sequence $x_1, x_2 \ldots x_n$ as the output from the statistical or simulation model. Also consider the threshold y_{jk} which is the k-th alternative instream requirement with respect to the j-th attribute (see Figure 3). The attribute value x_{jl}, $l = 1, n$ is assumed to be the

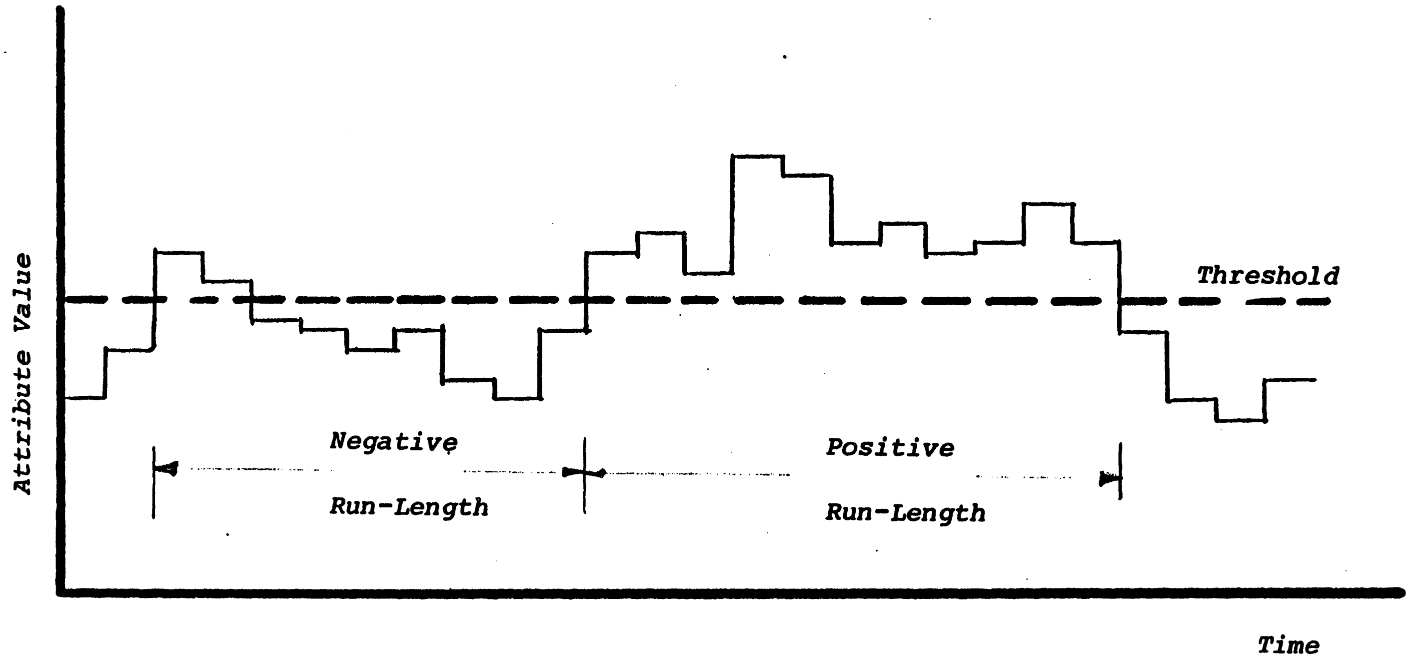

Figure 3. Hypothetic Time Sequence of Attribute

average value over the time interval, hence the bar graph nature of the time sequence. The time interval may be in days, weeks, month or years. For descriptive purposes, weekly averages are assumed. A negative run occurs when x_{jl} is less than y_{jk} during one or more interval while a positive run occurs otherwise. For example, a negative run is desirable when the attribute is BOD and is undesirable when the attribute is DO. Run characteristics such as run length, run sum, time between negative (positive) runs may be defined. These characteristics are random variables whose sample statistics may be computed. The statistics include the mean, standard deviation, range, maximum and minimum sum and skewness of each of the characteristics. Some of the statistics are used below, following Hashimoto, Stedinger and Loucks (1982), in order to define new parameters which are considered as measures of effectiveness of each vector of instream requirements. These parameters are the stream reliability, resiliency and vulnerability, and may be used, in addition to cost, benefits and others, as criteria for choosing between alternative instream requirement vectors. We define the variable Z_{ijk} as the value of the i-th measure for the j-th attribute and k-th alternative IRV. The index value for the reliability measure is i=1, for the resiliency measure is i=2 and for the vulnerability measure is i=3. These three terms are defined below in the context of this study.

<u>Reliability:</u> the probability that the stream meets the instream requirement for a given attribute. Given a generated sequence of this attribute, let

S_{jn} = number of weeks in the n-th year that the stream requirement is met with respect to the j-th attribute.

s = number of weeks in the year = 52.

Reliability = Z_{1jk}

mean fraction of time over the simulated N years that requirements for attribute j has been met.

$$Z_{1jk} = \Sigma(S_{jn}/s) \qquad (3.2)$$

<u>Resiliency:</u> describes how quickly the stream is likely to recover from failure (deficiency with respect to a given attribute) once failure has occurred. Let u_t indicate a transition from a satisfactory to an unsatisfactory state; otherwise $u_t = 0$ for the t-th transition. The variable, T, is the total number of transitions.

Then:

$$P = \text{transition probability} = \Sigma u_t$$

Let the average sojourn time in the unsatisfactory state be τ; then the expected value of τ is

$$E(\tau) = (1-Z_{1jk})/P \qquad (3.3)$$

= average number of periods failure is expected to last.

$$\text{Resiliency, } Z_{2jk} = 1/(E(\tau)$$
$$= (P/(1-Z_{1jk})) \qquad (3.4)$$

<u>Vulnerability:</u> describes the likely consequence of failure and is defined as the expected maximum severity of a sojourn into a sequence of failure weeks

$$Z_{3jk} = (\Sigma((y-x_l)/y)^{\beta})(1/L) \qquad (3.5)$$

where $(y-x_l)$ is the maximum deficit that occurred in the l-th sequence of failure weeks, β defines the shape of the consequence (loss) function and y is the instream requirement.

The output from this section are values of the vector of the measures of effectiveness for each attribute. They are used in the decision analysis as choice criteria.

<u>Expressions for Event Based Simulation</u>: The event based stormwater model described in Section 3.2 may be used to obtain an optimal IRV for a stream segment under risk from overflows due to a storm event over an urbanized catchment. Recall that a special feature of this model is that storm duration and magnitude can be sampled from the bivariate empirical distribution developed from the National Weather Service published rainfall intensity values for the locality. In a design scenario, the duration and magnitude are specified for the design return period and the impact simulated in the context of violations of elements of the IRV. The interest in this section is the decision scenario of the choice of an optimum IRV.

The procedure is to run a predetermined number of replications of the model, sampling from the joint empirical distribution of rainfall magnitude and duration. Each replication gives the magnitude and duration of exceedance/deficit. The exceedance duration and magnitude are given as the positive run-length and run-sum respectively. Correspondingly, the deficit duration and magnitude are given as the negative run-length and run-sum, respectively.

The expressions for reliability, resiliency and vulnerability are obtained in this case in the context of a single event. The sample statistics of the derived variables (negative run-sum, positive run-sum) are computed over the number of replications. The implicit assumption is that the storm events are independent. Some of these events cause overflow of treatment devices. Such overflows, by virtue of their pollution content, increase instream problems when they empty into a stream reach at risk.

The Reliability, Z_{1jk}, is given by

$$Z_{1jk} = S_j/N \qquad (3.6)$$

where S_{jk} is the number of times the stream meets the k-th candidate instream requirement with respect to the j-th attribute.

The Resiliency, Z_{2jk}, is given by

$$Z_{2jk} = P/(1-Z_{1jk}) \qquad (3.7)$$

where P = transition probability

$= (\Sigma u_t)/T \qquad (3.8)$

and u_t = 1 if the t-th transition is from a satisfactory to an unsatisfactory state;

= 0 otherwise.

T = total number of transitions in the N replications.

The Vulnerability, Z_{3jk}, is given by

$$Z_{3jk} = \Sigma((y-x_l)/y)^{\beta} \tag{3.9}$$

where $(y-x_l)$ is the maximum deficit during the l-th failure event and L is the total number of failure events. Failure is considered to occur when the stream does not meet the instream requirement.

3.3.2 Instream Requirement Vector

The instream requirement vector (IRV) is considered to be synonymous with a set of stream standards for corresponding attributes. The classification of the stream then depends on the associated IRV. The IRV gives municipalities, industries and other stream users specific goals to attain when treating waste waters that are returned to receiving streams. It provides the regulatory agencies a basis for uniform regulation of the use of the stream segment. The South Carolina Department of Health and Environmental Control (DHEC), which is the state's regulatory agency, has developed a water classification and standards system. In this system, all fresh water streams fall into one of three classes - B, A-TROUT , A, or AA. The standards corresponding to class B are the least stringent.

Horton (1965), has recommended index number systems for rating stream quality. The usual argument against stream standards is that they are often interpreted in absolute terms which may lead to irrational situations. The following illustrative example may be given. A regulatory agency may stipulate that a class A stream segment has BOD concentration of less than 12.0 mg/l and another agency may use a BOD concentration of 10.0 mg/l. Both agencies may designate a class A stream segment as suitable for recreation. The result would be that a stream with a 12.0 mg/l BOD concentration could be used for recreation in one area and not in another.

Recent systems of classifications have recognized the limitation of such number specific standards and have provided classifications in terms of ranges. The allowable range of pH may be $6.0 \le pH \le 8.5$ while the allowable increase of stream temperature above the ambient may be $8^{o}F \le T \le 10^{o}F$. Using the post-sensitivity analysis option of the LP algorithm, the GP formulation may be used to determine whether the optimum vector remains optimum over the range of parameter values specified in the standard.

For the purposes of implementing the methodology of this study, we shall arbitrarily assume a set of four gradations in stream attributes.

3.3.3 <u>Decision Analysis</u>

The Decision Analysis for the optimum vector of instream requirements is performed by constructing the decision matrix, given in Table 3. This table is general as

Table 3. Multiattribute Multimeasure Decision Matrix

Measure i	Reliability 1			Resiliency 2			Vulnerability 3			
Attribute j	FLOW 1	DO 2	BOD 3	FLOW 1	DO 2	BOD 3	FLOW 1	DO 2	BOD 3	Figure of Merit
Alternative										
y_1	.	z_{121}	.	.	.	.	.	z_{321}	.	W_1
.	.	.	.	.	.	.	.	.	.	.
.	.	.	.	.	.	.	.	.	.	.
.	.	.	.	.	.	.	.	.	.	.
y_k	.	.	.	.	z_{22k}	.	.	.	.	W_k
.	.	.	.	.	.	.	.	.	.	.
.	.	.	.	.	.	.	.	.	.	.
y_K	.	.	.	.	.	.	.	.	.	W_K

y_k = The k-th alternative

δ_{ij} = The importance weight of the i-th measure and j-th attribute

z_{ijk} = The normalized value of the i-th measure for the j-th attribute and the k-th alternative

W_k = The figure of merit of alternative k

$= \sum_i \sum_j z_{ijk} \delta_{ij}$

it assumes that all three measures of effectiveness are being used for the analysis. This contrasts with the single measure illustrated in Table 1. Each alternative is a vector of possible instream requirements with respect to the individual stream attribute. Thus $Y_k = [y_{1k}, y_{2k}, y_{3k}, \ldots y_{jk}]$ where y_{jk} is the k-th possible requirement for the j-th attribute. The entry, Z_{ijk} is the value of the i-th criterion for the j-th stream attribute and k-th possible instream requirement and constitutes the outcome.

The simple additive weighting method requires comparable scale for all entries (elements) in the decision matrix. Comparable scale is obtained by using Eq. (3.10) for the reliability and the resiliency entries since these indicate positive effects. Equation (3.11), in contrast, is used for vulnerability entries.

$$Z_{ijk} = Z_{ijk}/Z^{*} \qquad i = 1,2 \qquad (3.10)$$

where Z^{*} is the largest entry in the column.

$$Z_{ijk} = Z^{*}/Z_{ijk} \qquad i = 3 \qquad (3.11)$$

where Z^{*} is the smallest entry in the column. The weighted outcome over all measures and attributes for the k-th IRV is

$$W(k) = \Sigma\ \Sigma\ \delta_{ij} Z_{ijk} \qquad (3.12)$$

where δ_{ij} is the importance weight of the i-th measure and j-th attribute. Then the most preferred alternative, A^*, i.e. IRV, is selected such that

$$A^* = \{A_k \mid \max_{\text{for all } k} W(k)\} \qquad (3.13)$$

A FORTRAN computer program for automating the determination of A^* is given in Appendix B.

For the Goal Programming formulation the constraint equation is written as:

$$Z_{ijk} \cdot Y_{jk} - (N_{yk} - g_{yk}) = Y_j^* \qquad (3.14)$$

where Z_{ijk} is the normalized measure of effectiveness , Y_{jk} is the k-th alternative instream requirement for the j-th attribute (objective). The parameter Y_j^* is the most desirable stream condition for the j-th attribute. This condition may be the class AA stream standard for the attribute. The variables h_{ijk} and g_{ijk} are, respectively, the positive and negative deviation from the instream requirement. Further normalizing by dividing by Y_i^* gives

$$(Z_{ijk})(g_{jk}) - (h_{ijk} - g_{ijk}) = 1 \qquad (3.15)$$

where $g_{jk} = (Y_{jk}/Y_j^*)$, and h and g are normalized deviations. The above goal constraint equation must be written for all attributes and measures of effectiveness for each alternative instream requirement vector. The objective function remains as given in Eq. (2.4).

IV APPLICATION

Statistical modeling and mathematical simulation have been described in Chapter 3 for constructing sequences of stream attributes. The simulation approach is event-based and provides a time series of flow, BOD and DO at points downstream from an outfall. In contrast, given the attribute sample statistics, the statistical model may be used to generate the sequence of corresponding attribute values. Four possible combinations of analysis within the methodology include:

a) Event-based simulation to obtain quantity and quality impacts downstream and using the value-based approach to obtain the IRV;

b) Same as above but using the goal programming approach to obtain the IRV;

c) The statistical model to generate sequences of stream attributes and the value-based approach to obtain the IRV;

d) The statistical model to generate sequences of stream attributes and the Goal Programming approach to obtain the IRV.

As stated, the primary thrust of the study is to develop and demonstrate a multiattribute - multimeasure method for stipulating a vector of instream requirements. Such a demonstration is provided in Section 4.2 for case c. The available data for the selected stream is described in the next section.

4.1 AVAILABLE DATA

Generally stated, the attributes with respect to which a stream segment may be assigned to a class include:

(a) physical, chemical and biological characteristics;

(b) use of lands adjacent to the stream segment;

(c) current level of waste reception;

(d) current and future uses of the segment;

(e) economic incentive to improve stream quality;

The South Carolina Water Resources Commission (1983) has collected some of the above mentioned data for selected stream segments in South Carolina. The choice of target stream segments is based on the availability of data. The Bureau of Water Pollution Control of the State Department of Health and Environmental Control has developed a sampling network for water quality (physical, chemical, biological). The network is comprised of primary and secondary sampling stations for each of six regions of the State. The current interest is in the Greenville zone which includes the Tyger River. There are 29 stations in this region of which station B-008 (see Figure 4) is part of the National Basic Ambient Monitoring Program. The primary stations were sampled once every month at different days in the month, while the secondary stations are sampled even less frequently.

The available data were acquired from the STORET repository. Table 4 shows the data from the primary sampling station 21SC60 WQ <u>B-008</u> for June, 1980 through October, 1984. The water quantity and quality attributes and

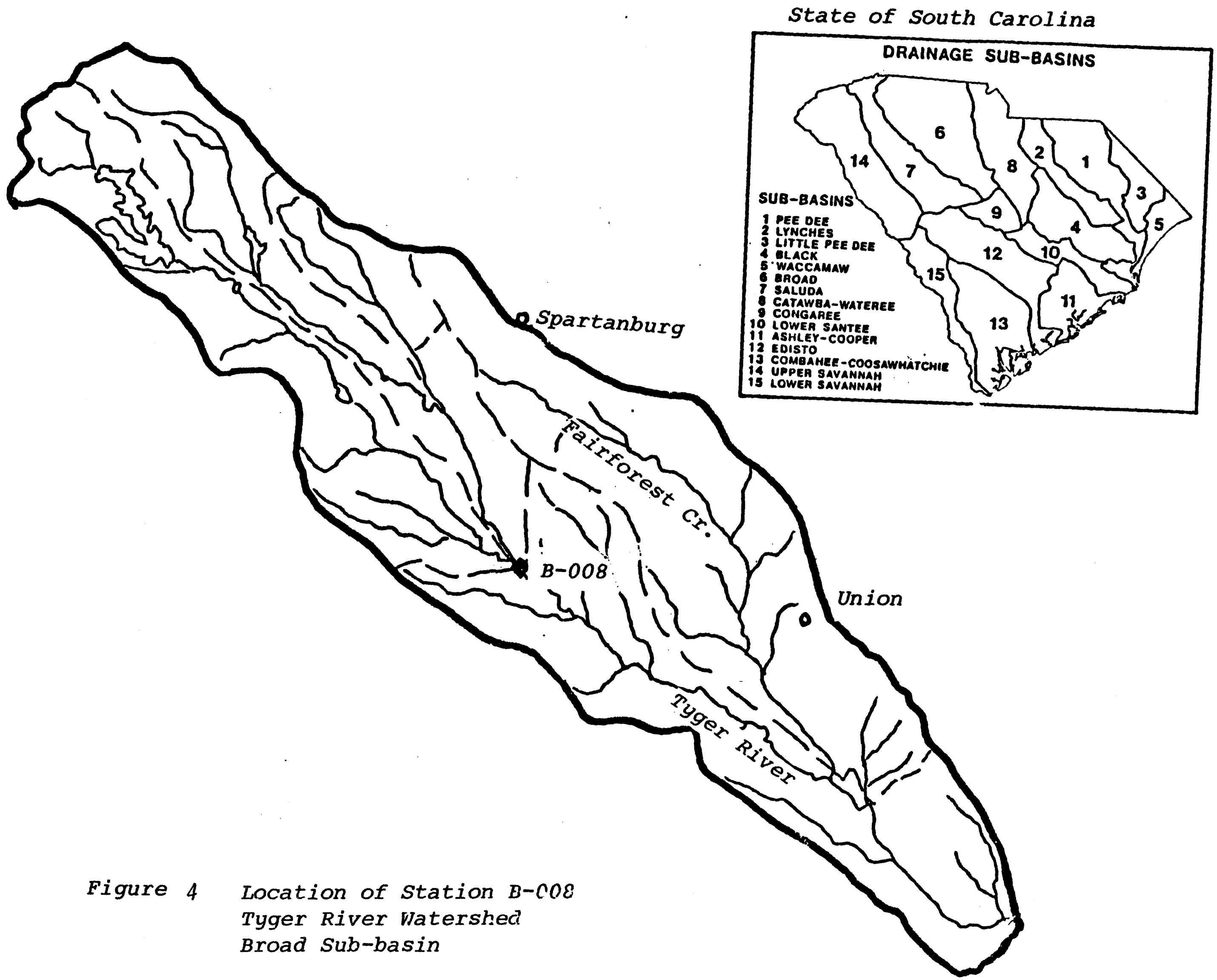

Figure 4 Location of Station B-008
Tyger River Watershed
Broad Sub-basin

Table 4. Water Quality and Quantity Data
(Station B-008 , Tyger River at S-42-50)

Date	Water Temp	Stream Flow	DO	5-Day BOD
800513	26.5	9999	8.8	2.09
800613	23.0	9999	8.9	1.19
800718	31.0	9999	7.8	1.39
800813	36.0	9999	8.1	1.69
800930	19.0	212	9.2	2.59
801021	17.0	422	9.6	1.89
801125	12.5	724	10.7	3.50
801215	7.5	410	12.0	2.19
810120	6.0	338	13.2	1.00
810212	30.0	108	12.3	4.39
810320	14.0	349	10.9	2.00
810423	28.0	333	9.3	1.90
810515	18.0	268	8.6	1.79
810622	21.0	174	7.1	1.89
810710	36.0	147	8.0	1.59
810804	34.0	167	7.9	1.20
810922	30.0	108	9.4	2.89
811007	18.0	70	8.8	2.00
811120	19.0	160	11.4	2.89
811210	-1.0	204	12.9	1.80
820108	8.0	944	11.2	2.59
820222	27.0	544	10.4	2.07
820319	99.9	560	11.5	1.70
820402	99.9	410	10.0	4.00
820526	28.0	688	7.9	2.80
820608	31.0	363	8.2	1.40
820719	33.0	389	7.5	1.19
820804	99.9	560	7.8	1.50
820916	25.0	213	8.1	1.69
821025	12.0	233	10.7	2.39
821126	11.0	275	10.7	2.30
821209	16.0	490	10.9	1.39
830110	9.0	375	11.4	1.69
830215	9.0	1540	11.8	3.40
830302	20.0	524	11.1	1.69
830302	99.9	9999	99.9	99.99
830406	26.0	730	9.7	2.00
830516	23.0	504	8.5	2.87
830608	24.0	456	8.0	2.27
830714	32.0	245	8.2	1.59
830805	32.0	217	8.7	1.59
830907	30.0	241	8.0	1.30
831013	22.5	340	8.5	2.50
831110	19.0	205	9.8	2.90
831229	5.0	953	12.5	2.09
840112	5.0	9999	12.6	.40
840216	12.0	9999	11.0	2.19
840315	23.0	9999	10.6	1.60
840417	15.0	9999	9.9	1.09
840530	17.0	9999	8.6	2.40
840622	21.0	9999	7.4	2.00
840716	99.9	9999	99.9	99.99
840813	15.0	9999	7.8	3.00
840919	21.0	9999	9.6	1.60
841011	24.0	9999	9.0	2.00

their measurement units are: water temperature (^{o}C), stream flow (cfs), DO (mg/L), and 5-day BOD (mg/L). Tables 5 and 6 show the statistics of the data containing 53 records. Some attributes were not sampled and hence were missing from some records. Missing data in Table 4 are indicated by 9999 or 99.99. Tables 5 and 6 also show how many observations were recorded for each attribute.

The segment immediately upstream from station B-008 is considered for the purpose of illustrating the methodology. General descriptions of the hydrology, economy, population and water use statistics on 15 subbasins in the State are given in the State Water Assessment (SCWRC, 1983).

4.2 APPLICATION OF THE VALUE BASED APPROACH

The Decision Analysis Program developed for implementing the Value-Based approach is given in Appendix B. The program accepts four sets of input data.

The <u>first</u> set comprise the integer constants NOBS, NVAR, NVEC and NEFF. NOBS is the number observations generated to constitute the sequence of each attribute, NVAR is the number of attributes, NVEC is the number of candidate Instream Requirement Vectors, and NEFF is the number of the measures of effectiveness.

The <u>second</u> set of input data comprise the importance (priority) weights for the attributes denoted by (b(j), j=1, NVAR), and the importance (priority) weights for the measures denoted by (a(i), i=1 NEFF).

Table 5. Sample Statistics of Data
(Station B-008)

	TEMP	FLOW	DO	BOD
NO. OF CASES*	50	40	53	53
MEAN	20.420	404.825	9.670	2.058
STANDARD DEV	9.116	282.880	1.638	0.755
SKEWNESS	-0.233	1.931	0.412	0.838

*The difference between total observations and the number of cases represents missing data.

Table 6. Correlation Matrix of Data
(Station B-008)

	TEMP	FLOW	DO	BOD
TEMP	1.000			
FLOW	-0.360	1.000		
DO	-0.758	0.313	1.000	
BOD	-0.129	0.255	0.277	1.000

The third set of input data include the mean, standard deviation and autocorrelation coefficient of each attribute and are denoted by AV(j), STD(j) and R(j) for the j-th attribute.

The fourth set of input data are the vector of candidate instream requirements denoted by $(\text{xcrit}(j),\ j=1,\ \text{NVAR})_k$ for the k-th candidate vector.

For this illustrative example the attributes considered are the Flow, DO and the 5-day BOD. The sample statistics for these attributes are taken from Table 5. Table 7 shows the order in which the input data were entered. Table 8 shows the output from the program INSTRM. The last column contains the value of the Figure of Merit (FOM) for each candidate IRV. Alternative 3 gave the highest FOM value at 0.46, and therefore is optimum. The corresponding values of the elements of the vector, i.e. the stream standards for this example, are FLOW = 350 cfs., DO = 6.0 mg/L and BOD = 3.5 mg/L. We hasten to add that these IRV values have been obtained based on arbitrarily selected values for the candidate IRV's.

Table 7. Input Data For Program INSTRM

Input set one	NOBS	NVAR	NVEC	NEFF
	1001	3	5	3
Input set two				
	RELIAB	RESIL	VULNER	
Measure weights	0.4	0.3	0.3	
	FLOW	DO	BOD	
Attribute weights	0.4	0.3	0.3	
Input set three (sample statistics)				
Mean	404.8	9.67	2.06	
Standard Deviation	282.9	1.64	0.76	
Serial corr. coef.	0.35	0.15	0.10	
Input set four (IRV Alternatives)				
1	404.8	9.67	2.06	
2	460.0	8.50	1.50	
3	350.0	6.00	3.50	
4	235.0	7.00	2.30	
5	500.0	10.50	1.80	

Table 8. Output From Program INSTRM

CANDI-DATE	FLOW (j=1)			DO (j=2)			BOD (j=3)			FOM
IRV's	i=1	2	3	1	2	3	1	2	3	
k=1	.65	.68	.57	.42	.44	.39	.03	.60	.15	.20
2	.55	.98	1.00	.36	.92	1.00	.03	.00	.03	.32
3	.78	1.25	.03	.55	11.00	.22	.02	.00	1.00	.46
4	.90	1.20	.47	.73	3.67	.34	.01	.00	.14	.29
5	.43	.35	.78	.32	.28	.55	.04	.76	.05	.22

THE OPTIMUM IRV IS 3 WITH AN FOM VALUE OF 0.46

FLOW = 350 cfs DO = 6.0 mg/l BOD = 3.50 mg/l

V. SUMMARY AND CONCLUSIONS

A methodology was developed for establishing instream requirements for multiple attributes. The instream requirement vector was evaluated using multiple measures of effectiveness. The Expected Value and the Goal Programming formulations were used, separately, to model the multiple attribute aspect of the methodology. The application of these formulations was illustrated with a case study in water resources development.

The methodology may be applied to target stream segments that are used for industrial, municipal, agricultural and thermoelectric power use, hydroelectric power generation, commercial and recreational fishing, navigation, maintenance of endangered species, and waste water assimilation. The analyst only needs to identify an appropriate set of attributes to represent the uses of interest. Possible attributes for these uses include stream withdrawal rates and discharge, DO, BOD, sediment concentrations and biota counts. The discharge, DO and BOD were used as attributes in the illustrative example.

A sequence of each attribute was generated using the First Order Markov model. Sample statistics for use in the model were calculated from data collected by DHEC. These data were collected as part of the National Basic Ambient Monitoring Program and are stored in the STORET repository. A planning type Urban Runoff Quality and Quantity model which may be used to generate stream attributes was also developed.

This model is a significant modification of the Simplified Storm Water Management Model developed by Lager et. al. In addition to its use for generating sequences of stream attributes, the model may be used for sizing detention ponds for storm water treatment and open channels for drainage purposes. A significant feature of the model is that it includes a subroutine that constructs the joint empirical distribution of the duration and magnitude of storm events. Given a return period, a second routine inverts the above joint distribution to obtain the corresponding design duration and volume. It assumes one of two standardized storm patterns to obtain the design hyetograph.

Crossing level analysis was performed on each generated sequence with the threshold being the candidate attribute instream requirement. The output from this analysis were the computed values of the stream vulnerability, reliability and resiliency which are used as measures of the effectiveness of the requirement. Other measures, such as costs incurred or benefits accrued in maintaining the vector of instream requirement, may as well be used.

These output values, after conversion to comparable scales, were the entries in the multiattribute - multimeasure matrix for the selection of the optimum instream requirement vector. The conversion was obtained by dividing each entry in a reliability column by the largest entry in the column, each entry in a resiliency column by the largest entry in the column and each entry in a vulnerability column by the smallest entry in the column. The Simple Additive Weighting method was used to obtain the weighted outcome for each alternative instream requirement vector. The alternative with the highest weighted outcome is the most preferred.

The product includes two computer programs. The first program is the model described above to simulate, for given design hyetographs, the impact of detention storage overflows on the stream quality at downstream locations. The second program automates the decision analysis for the selection of the instream requirement. The product can serve as a decision tool for constructing irrigation systems, developing municipal water withdrawal schedules and identifying instream requirements.

VI REFERENCES

Bloxham, W. M., "Low-Flow Characteristics of Ungaged Stream in the Piedmont and Lower Coastal Plain of South Carolina," South Carolina Water Resources Commission, Report No. 14, 1981.

Bovee, K. D., "A Guide to Stream Habital-Analysis Using the Instream Flow Incremental Methodology," Office of Biological Service, Rep. FWS/OBS-82/26, Paper No. 12, 1982

Daubert, J. T., Young, R. L., and Gray, Lee S., "Economic Benefits from In-stream Flow in a Colorado Mountain Stream," Completion Report No. 91, Colorado Water Resources Research Institute, CSU, Fort Collings, 1979.

de Kozlowski, S. J., "Instream Flow Study Phase I: Identification of Priority Listing of Streams in South Carolina for which Minimum Flow Levels Need to be Established," South Carolina Water Resources Commission, Report No. 149, 1985.

Erskine, T. C. and Shih, C. S., "Subjective Decision-Making for Water Resources Development," 41st National ORSA Meeting, New Orleans, LA, 1972.

Fiering, M. B. and Jackson, B. A., "Synthetic Streamflow," American Geophysical Union, Water Resources Monograph No. 1, 1971.

Governor's State Water Law Review Committee, "Report and Recommendations" to the Governor of South Carolina, Office of the Governor, Columbia, SC, 1982.

Hashimoto, T., Stedinger, J. R., and Loucks, D. P., "Reliability, Resiliency and Vulnerability Criteria for Water Resource System Performance Evaluation", <u>Water Resources Research,</u> Vol. 18(1), pp. 14-20, 1982.

Horton, R. K., "An Index-Number System for Rating Water Quality", <u>Journal of the Water Pollution Control Federation,</u> Vol. 37(3), pp. 300-307, 1965.

Lager, J. A., Didriksson, T., and Ott, G. B., "Development and Application of a Simplified Storm Water Management Model," Report EPA-600/2-76-218, Environmental Protection Agency, Washington, D. C., 1976.

Medina, M. A. Jr., " Level III: Receiving Water Quality Modeling for Urban Stormwater Management," EPA-600/2-79-100, Environmental Protection Agency, Washington, D. C., 1979.

Medina, M. A. Jr., "Hydrologic and Water Quality Modeling for In-stream Flow Strategies," Water Resources Research Institute, University of North Carolina, Raleigh, NC, Report No. 183, 1982.

Medina, M. A. Jr., " An Integrated Methodology for Instream Flow Strategies," Water Resources Research Institute, University of North Carolina, Raleigh, NC, Report No. 210, 1983.

Narayanan, R., et. al., "An Economic Evaluation of Benefits and Cost of Maintaining Instream Flows," Utah Water Research Laboratory, Utah State University, Logan, Water Resources Planning Series UWRL/P-83/04, 1983.

Sale, M. J., Grill Jr., E. D., and Herricks, E. E., "An Approach to Optimizing Reservoir Operation for Downstream Aquatic Resources," Water Resources Research, Vol. 18(4), pp. 705-712, 1982.

Singh, K. P., and Stall, J. B., "Hydrology of 7 day 10-year Low Flow," Journal of Hydraulics Division, ASCE , Vol. 100, (HY12), 1974.

South Carolina Water Resources Commission, " South Carolina State Water Assessment," South Carolina Water Resources Commission, Report No. 140, Columbia, SC, 1983.

Wallace, J. R., Day, G. N., Howard, W. R., and Wiederkehr, K. J. "Methods of Low Flow Prediction for Small Georgia Streams," Environmental Resources Center, Georgia Institute of Technology, Atlanta, GA ERC 03-80, 1980.

Walsh, R. G., "Empirical Application of a Model for Estimating the Recreation Value of Water in Reservoirs Compared to In-stream Flow," Water Resources Research Institute, Colorado State Univertity, Fort Collings, Report No. 103, 1980.

Walsh, R. G., Ericson, R. K., Arosteguy, D. J., and Henson, M. P., "An Empirical Application of a Model for Estimating the Recreation Value of In-stream Flow,"Water Resources Research institute, Colorado State University, Fort Collings, Report No. 101, 1980.

APPENDIX A

Data For Watershed-River Response Program

1. <u>Storm Analysis Data Entry:</u>

The data includes two lines for each day of record.

Line	Columns	Description	Variable Name
1	07-08	Year	NY(i)
	09-10	Month	MO(i)
	11-12	Day	DD(i)
	13	Switch indicating time of day	NX(i)
	14-16	Quantity of rain that occurred in hr 1	FR(i,1)

	47-49	Quantity of rain that occured in hr 12	FR(i,12)
2	14-16	Quantity of rain that occurred in hr 13	FR(i,13)

	47-49	Quantity of rain that occurred in hr 24	FR(i,24)
	55-56	Day of next recorded rainfall	NX(i)

2. <u>Example of input to the Design Storm program</u>

```
50.00
82010100.000.030.000.000.000.020.020.000.030.000.000.00
       0.000.000.000.000.000.000.000.000.000.000.000.00  2
82010200.000.000.000.000.000.000.000.000.010.000.000.00
       0.000.000.000.000.000.000.000.000.000.000.000.00  4
```

3. <u>Example of the output from the Design Storm program</u>

Sample Statistics of Rainfall:

	Average	Std. Dev.	Skew
Magnitude (in)	0.218	0.310	3.0
Duration (hr)	7.300	7.970	3.7

Cumulative Distribution Function:

Magnitude	CDF	Duration	CDF
0.00	0.000	0.000	0.000
0.275	0.604	6.625	0.479
0.550	0.840	13.250	0.785
0.825	0.931	19.875	0.917
1.100	0.965	26.500	0.958
1.375	0.986	33.125	0.979
1.650	0.993	39.750	0.986
1.925	0.993	46.375	0.993
2.200	0.993	53.000	0.993
2.475	1.000	59.625	1.000

Contingency Coefficient = 2.12

Rain Volume for 50 yr. Return Period = 1.02 inches
Rain Duration for 50 yr. Return Period = 36.87 hours

4. <u>Watershed Simulation Program</u>

Example for a hypothetical watershed :

<u>Watershed Characteristics</u>

Subarea	Area (mi^2)	Runoff CN	Tc (hrs)	Tp (hrs)	Qp (cfs)
1	10.0	75.0	2.0	1.333	3631.
2	12.0	68.0	2.7	1.8	3227.
3	7.8	80.0	1.5	1.0	3775.
4	11.0	71.0	3.0	2.0	2662.
5	20.0	72.0	6.1	4.066	2381.

<u>Channel Characteristics</u>

Channel Reach	Length (ft)	Routing Coefficient
003	2000	0.40
006	1800	0.45
009	2700	0.30

5. Watershed Simulation Data Entry:

1- Each time a runnoff hydrograph from a certain subarea is to be calculated the following data should be entered:

On the first card : If Indcl is -1 or 0, the calculated hydrograph is to be routed, subroutine ROUTE is called. If Indcl is 1, the calculated hydrograph is to be added to the hydrograph currently in storage-subroutine ADD is called. If Indcl is 5,the hydrograph in storage is the final hydrograph signifying the end of data.

On the second card: N5, which is the number of sections into which the subarea being analyzed is divided depends on the variations in the soil properties and soil cover.

On the third card:A,Tp,Qp,Dc,(CN(i),CU(i),PERCENT(i),i=1,N5)

A=basin area in square miles.
Tp=total time to peak in hours.
Qp=unit hydrograph peak flow in cfs.
Dc=percent of directly connected impervious area.
CN(i)=curve number for section i of the subarea being analyzed.
Cu(i)=initial abstraction coefficient for section i.
percent(i)=section area/Total area*100.

2 - Each time subroutine ROUTE is called the following data should be entered:

On the first card: Qo,C

Qo=outflow at time 0.
C=routing coefficient of the channel through which the runoff hydrograph is to be routed.

On the second card: Indc2

If Indc2 is -1 or 0 the routed hydrograph is to be stored. If Indic2 is 1 the routed hydrograph is to be added to another hydrograph. If an added hydrograph is to be routed there is no need for Indc2. The second card would then be ommitted.

3- Each time the addition subroutine is called the following data should be entered.

On the first card: Indc3

If Indc3=+1, the resulting hydrograph in storage, after subroutine ADD is called, will be routed to another point in the watershed. This means that the second card has the routing data on it.

If Indc3=-1 or 0, the resulting hydrograph is to be stored and the next card has Indcl on it. For the previously given example the input data should be entered as shown below:

Line	Data	Explanation
1	-1	(Indc1)
2	1	(N5)
3	7.8 1.0 3775. 0.0 80. 0.2 100.	(Subarea 3 Char.)
4	0.0 0.45	(Routing Data)
5	-1	(Indc2)
6	1	(Indc1)
7	1	(N5)
8	11. 2.0 2662. 0.0 71. 0.2 100.	(Subarea 4 Char.)
9	-1	(Indc3)
10	-1	(Indc1)
11	1	(N5)
12	10.1.333 3631. 0.0 75. 0.2 100.	(Subarea 1 Char.)
13	0.0 0.4	(Routing Data)
14	1	(Indc2)
15	-1	(Indc3)
16	1	(Indc1)
17	1	(N5)
19	12 1.8 3227. 0.0 68. 0.2 100	(Subarea 2 Char.)
20	1	(Indc3)
21	0.0 0.3	(routing data)
22	1	(Indc1)
23	1	(N5)
24	20. 4.066 2381. 0.0 72. 0.2 100.	(Subarea 5 Char.)
26	-1	(Indc3)
27	5	(Indc1)

6. Output from the Watershed Simulation Program

Percent-Time	Rain-Increment
2	0.0768000
3	0.0428000
5	0.0864000
7	0.0864000
8	0.0432000
10	0.0944000
12	0.0960000
13	0.0480000
15	0.0980001
17	0.1055998
18	0.0528002
20	0.1116002
22	0.1767991
23	0.1024008
25	0.0288000
27	0.1435995
28	0.0768003
30	0.1616001
32	0.1800003
33	0.0879993
35	0.2200003

7. Output from the Watershed Simulation Program

Time(hrs)	Rainfall(ins)	Rainfall Excess
0.2500000	0.0000000	0.0000000
0.5000000	0.0768000	0.0000000
0.7500000	0.0428000	0.0000000
1.0000000	0.0864000	0.0000000
1.2500000	0.0864000	0.0000000
1.5000000	0.0432000	0.0000000
1.7500000	0.0944000	0.0000000
2.0000000	0.0960000	0.0000000
2.2500000	0.0480000	0.0000000
2.5000000	0.0980001	0.0000000
2.7500000	0.1055998	0.0200000
3.0000000	0.0528002	0.0100000
3.2500000	0.1116002	0.0300000
3.5000000	0.1767991	0.0600000
3.7500000	0.1024008	0.0400000
4.0000000	0.0288000	0.0100000
4.2500000	0.1435995	0.0600000
4.5000000	0.0768003	0.0400000
4.7500000	0.1616001	0.0800000
5.0000000	0.1800003	0.1000000
5.2500000	0.0879993	0.0500000
5.5000000	0.2200003	0.1300000
5.7500000	0.3023996	0.2000000

8. Output from the Watershed Simulation Program

Unit Hydrograph Ordinates

Time(hrs)	Discharge(cfs)
0.0000000	0.0000000
0.2500000	53.5286200
0.5000000	1302.6700000
0.7500000	3142.8300000
1.0000000	3775.0020000
1.2500000	3380.8640000
1.5000000	2623.0140000
1.7500000	1886.7770000
2.0000000	1302.6660000
2.2500000	879.9377000
2.5000000	588.0693000
2.7500000	391.4770000
3.0000000	260.6835000
3.2500000	174.1022000
3.5000000	116.8171000
3.7500000	78.8270500
4.0000000	53.5282200
4.2500000	36.5922000
4.5000000	25.1862100
4.7500000	17.4552600
5.0000000	12.1804300
5.2500000	8.5571420

9. Output from the Watershed Simulation Program

Design Hydrograph

Time(hrs)	Flow(cfs)
0.0000000	0.0000000
0.2500000	0.0000000
0.5000000	0.0000000
0.5000000	0.0000000
0.7500000	0.0000000
1.0000000	0.0000000
1.2500000	0.0000000
1.5000000	0.0000000
1.7500000	0.0000000
2.0000000	0.0000000
2.2500000	0.0000000
2.5000000	0.0000000
2.7500000	0.8924348
3.0000000	22.2976500
3.2500000	67.9883700
3.5000000	136.2362000
3.7500000	260.2180000
4.0000000	413.6013000
4.2500000	506.8215000
4.5000000	571.2761000
4.7500000	650.6645000
5.0000000	762.2763000

10. <u>River Response Data Entry</u>:

Line	Variable	Description
1	FFLBS	Rate of accumulation of BOD,lb/h
	DWHRS	Time since the last rainfall,hrs
	WWBOD	Rainfall BOD,lb/cu. ft.
	RQ	Original flow in the the receiving water, cfs
	RBOD	Original receiving water BOD,lb/cu.ft.
	RTEMP	Original receiving water temp., ^oC.
	RDO	Original receiving water DO,lb/cu. ft.
2	G	Gravity constant
	RO	Receiving water density
	XK13	Biochemical oxydation rate and sedimentation rate coef. for carboneous BOD
3	NT	Number of times the BOD and DOD are to be calculated at a certain distance downstream from the point of discharge of the watershed runoff.
4	NX	Number of points along the stream where BOD-DOD concentration are to be calculated.
	X(i)	Distance downstream from the point of discharge where BOD-DO concentrations are to be calculated.
5	S(i)	Slope of the channel in ft/ft
	ZN(i)	Manning roughness coefficient.

11. <u>Example of input into the River Response routine</u>

```
200. 6.22 50 150 27 15 7
32.2 1.97 0.1 0.1 0.9 0.1
40  3 300 1100 15840
.005 .05 .006 .05 .006 .055
0.4
```

12. Output from the Watershed-River Response Program

Time(hrs)	BOD (mg/l)	DO Deficit(mg/l)
1.0000000	28.6573700	2.9348010
2.0000000	51.8699400	2.0716540
3.0000000	45.8568200	1.2649090
4.0000000	40.4346600	1.1141550
5.0000000	23.5284200	0.9312893
6.0000000	17.5233600	1.0175890
7.0000000	15.8305400	1.1823500
8.0000000	15.5951200	1.3492940
9.0000000	16.0789900	1.5136390
10.0000000	17.0315800	1.6833070
11.0000000	18.3329600	1.8647160
12.0000000	18.9126500	2.0608750
13.0000000	21.6928100	2.2712390
14.0000000	23.6963500	2.4953390
15.0000000	26.3438800	2.7445700
16.0000000	30.0483300	3.0385740
17.0000000	35.0260300	3.3924510
18.0000000	41.2606500	3.8017420

13. <u>Input to the Peak Flow Program</u>

```
-1
 1
 7.8 1.0 3775. 0.0 80. 0.2 100.
 0.0 0.45
-1
 1
 1
 11. 2.0 2662. 0.0 71. 0.2 100.
-1
-1
 1
 10. 1.333 3631. 0.0 75. 0.2 100.
 0.0 0.4
 1
-1
 1
 1
 12. 1.8 3227. 0.0 68. 0.2 100.
 1
 0.0 0.3
 1
 1
 20. 4.066 2381. 0.0 72. 0.2 100.
-1
 5
 0.5 50.
82010100.000.030.000.000.000.020.020.000.030.000.000.00
       0.000.000.000.000.000.000.000.000.000.000.000.00  2
82010200.000.000.000.000.000.000.000.000.010.000.000.00
       0.000.000.000.000.000.000.000.000.000.000.000.00  4
```

14. Output form the Peak Flow program

Sample Statistics

Average	Std. Dev.	Skew
152.3	341.7	2.1

Cumulative Distribution Function

Peak Flow(cfs)	CDF
0.00	0.000
224.80	0.765
449.60	0.882
674.80	0.882
899.20	0.882
1124.00	0.941
1348.80	1.000

Peak Flow for 50 yr. Return Period = 1047.8 cfs

APPENDIX B

Decision Analysis Program

```
c                           PROGRAM INSTRM.FOR
c                           ******************
c   The program performs a multi-attribute multi-measure
c decision analysis of stream water management system to
c obtain a set of Instream requirements that optimizes the
c conflicting uses of the stream segment. It reads in the
c set of input values from file 'INSTRM.DAT' and calls the
c subroutine GENE for generating the sequences of stream
c attributes. It calls subroutine XLEVEL to perform the
c crossing level analysis and computes the measures of effec
c -tiveness.
c
      dimension z(3,5,4),x(1001,5),av(5),sd(5),r(5)
      dimension FOM(4),xcrit(5),a(3),b(3)
      open (1,file='wrip.dat',status='old')
c
c     Read in Input data
      read(1,1) nobs,nvar,nvec,neff
    1 format(i4,3i2)
      read(1,2) (b(j),j=1,nvar)
      read(1,2) (a(i),i=1,neff)
    2 format(10f4.2)
      do 3 j=1,nvar
    3 read(1,4) av(j),sd(j),r(j)
    4 format(3f8.2)
      call GENE(av,sd,r,x,nobs,nvar)
      do 15 k=1,nvec
      read(1,10) (xcrit(j),j=1,nvar)
   10 format(10f8.2)
      call XLEVEL(x,xcrit,z,nobs,nvar,neff)
      write(*,17) ((z(i,j,k),j=1,nvar),i=1,neff)
   15 continue
   17 format(15f5.2)
      do 25 j=1,nvar
      z1=0.
      z2=0.
      z3=1000000.
      do 20 k=1,nvec
      if(z(1,j,k) .gt. z1) z1=z(1,j,k)
      if(z(2,j,k) .gt. z2) z2=z(2,j,k)
   20 if(z(3,j,k) .lt. z3) z3=z(3,j,k)
   25 continue
      do 40 k=1,nvec
      fom(k)=0.
      do 35 j=1,nvar
      z(1,j,k)=z(1,j,k)/z1
      z(2,j,k)=z(2,j,k)/z2
      z(3,j,k)=z3/z(3,j,k)
```

```
      do 30 i=1,3
   30 fom(k)=fom(k) + a(i)*z(i,j,k)
   35 fom(k)=b(j)*fom(k)
      write(*,*) k,fom(k)
   40 continue
      stop
      end
c
      SUBROUTINE GENE(AV,SD,R,X,NOBS,NVAR)
c
c    Routine generates NOBS 'observations' of each of the
c    NVAR variables (attributes) using the first order
c    Markov model given the sample statistics. 'Observations
c    are assumed NORMAL.
      dimension x(nobs,nvar),av(nvar),sd(nvar),r(nvar)
      iseed=12345
      do 150 j=1,nvar
      x(1,j)=av(j)
      do 100 i=2,nobs
      a=0.0
      do 50 in=1,12
      iseed=mod((25173*iseed + 13849),65536)
      rand=float(iseed)/65536.
   50 a=a + rand
      v=(a-6.0)
  100 x(i,j)=av(j)+r(j)*(x(i-1,j)-av(j))+sd(j)*sqrt(1-r(j)
     1*r(j))*v
  150 continue
      return
      end
      SUBROUTINE XLEVEL(X,XCR,Z,NOBS,NVAR,K)
c    Subroutine computes the RELIABILITY, z(1,j,k),
c    RESILIENCY, z(2,j,k) and VULNERABILITY, z(3,j,k) of the
c    j-th attribute and the k-th instream requirement vector.
c    DMAX  = maximum deficit that occured in the T-th
c            sequence of failure weeks, days, or hours.
c      b   = shape factor of the consequence (loss) function.
c   XCR(j) = instream requirement for j-th attribute.
c
      dimension x(1001,3),n(1001),d(1001),z(3,5,3),xcr(5)
      integer t,u(1500),tmax,b,it(1500)
c
      b=2
      do 100 j=1,nvar
      nsum=0
      do 10 i=1,nobs
      if(x(i,j) .gt. xcr(j)) n(i)=1
      if(x(i,j) .le. xcr(j)) n(i)=0
      d(i)=x(i,j) - xcr(j)
c     write(*,*) i,d(i)
   10 nsum=nsum+n(i)
      z(1,j,k)=float(nsum)/float(nobs)
      t=0
      p=0.
      do 20 i=2,nobs
      if(n(i-1) .eq. n(i) ) go to 20
```

```
      t=t+1
      it(t)=i-1
      tmax=t
      if(n(i-1) .eq. 1 .and. n(i) .eq. 0)  then
      u(t)=1
      else
      u(t)=0
      endif
      p=p+u(t)
   20 continue
      p=p/tmax
      z(2,j,k)=p/(1.-z(1,j,k))
      z(3,j,k)=0
      do 60 t=2,tmax
      l1=it(t-1)
      l2=it(t)
      if(d(l1) .gt. 0.) go to 40
      dmax=0
      do 30 l=l1,l2
   30 if(abs(d(l)) .gt. dmax ) dmax=abs(d(l))
      go to 50
      smax=0
   40 do 45 l=l1,l2
   45 if(d(l) .gt. smax) smax=d(l)
   50 continue
      z(3,j,k)=z(3,j,k) + (dmax/xcr(j))**b
   60 continue
  100 z(3,j,k)=z(3,j,k)/tmax
      return
      end
```