WATER RESOURCES ANALYSIS OF A MULTIOBJECTIVE DRAINAGE NETWORK IN THE INDIAN RIVER LAGOON BASIN

BY

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A THESIS PRESENTED TO THE GRADUATE SCHOOL
OF THE UNIVERSITY OF FLORIDA IN
PARTIAL FULFILLMENT OF THE REQUIREMENTS
FOR THE DEGREE OF MASTER OF SCIENCE

UNIVERSITY OF FLORIDA

1989

ACKNOWLEDGMENTS

My sincere thanks go to those individuals who provided technical guidance, financial support, a "free-thinking" work environment, and who interjected ideas and knowledge; all of which contributed toward the completion of this project. My time at the University of Florida and at the Kennedy Space Center has led to an interaction with a wide diversity of professionals, each addressing environmental problems from a different perspective. This "melting pot" arena has contributed immensely to my education.

Foremost, I would like to thank Professor James P. Heaney, my committee chairman, for guidance, technical expertise, and his encouragement to think. Thanks are also due to Professor Wayne C. Huber and Dr. C. Ross Hinkle, members of my committee, for their suggestions and review of the manuscript. A large debt is owed to Mr. Robert E. Dickinson of the Department of Environmental Engineering Sciences at the University of Florida for his invaluable assistance in understanding, applying, and modifying SWMM.

Funding for this research was provided by the National Aeronautics and Space Administration through the Bionetics Corporation of Kennedy Space Center, Florida, under contract number NAS10-10285. Dr. William M. Knott, III and Dr. Albert M. Koller, Jr., of the NASA Biomedical Operations and Research Office and Dr. C. Ross Hinkle of the Bionetics Corporation are gratefully acknowledged for their support in pursuing the issue and acquiring financial support for this study.

Special recognition is given to Mr. Carlton Hall and Mr. Mark Provancha, Bionetics Corporation, and Mr. Andreas Goetzfried, EG&G Florida. These individuals have provided time and encouragement and above all they provided valuable insight into the water resources and environment of Merritt Island, Florida.

A number of other individuals have been instrumental in various aspects of the project and have provided literature, data, discussion, and equipment. I acknowledge Mr. Kirby Key, Mr. Mario Busacca, and Mr. John Ryan of NASA; Dr. James Kanipe, Mr. Harry Morton, and Ms. Rena Gaines of EG&G Florida; and the Bionetics Ecological Programs staff.

Finally I wish to acknowledge my parents, Catherine and Berkley Bennett, for their love, support, patience, and sacrifice that enabled me to pursue an education.

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August 1989

Chairman: James P. Heaney
Major Department: Environmental Engineering Sciences

The slow demise of the Indian River Lagoon in East-Central Florida has been linked to drainage practices in the Basin. The feasibility of implementing a watershed control system at the Kennedy Space Center (KSC), Florida, to meet multiple water management objectives is presented. Site specific data were collected for the 1900 acre Industrial Area catchment in KSC. These data were used to calibrate and verify deterministic models of the watershed.

The U.S. Environmental Protection Agency's Storm Water Management Model (SWMM) RUNOFF module was selected to simulate total runoff flow, peak flow, groundwater seepage

to drainage channels, and storm event load of total suspended solids (TSS). The relative effectiveness of retrofitting the existing drainage network was evaluated using other modules of SWMM including the Storage/Treatment Block and the EXTRAN Block. Both continuous simulation and design storms were utilized in the analysis. Design criteria for Standard Project Flood, TSS removal, reduction in groundwater discharge, minimum and maximum water depths, and mean event discharge were used to evaluate performance.

A watershed control system that includes constructing weirs at channel outfalls with diversion of flow into coastal wetlands was found to be relatively effective in meeting many of the objectives. However, all the objectives could not be met without making compromises in the objectives. Due to analytical uncertainty, optimization of the watershed control system without actual performance data was not possible. Therefore, dynamic designs (i.e. adjustable water control structures) are recommended. Combining system performance data with dynamic simulation will allow fine tuning of the system. Study results suggest that this approach provides an effective watershed control strategy for the Indian River Lagoon Basin. Joint research/application projects will be required in the future.

CHAPTER I

INTRODUCTION

Historically, control of stormwater has consisted of collecting all runoff in gutters and discharging it into a conveyance system of storm sewers and channels which are tributary to a nearby stream, lake, estuary or ocean. A hydraulically efficient urban drainage system implies the use of storm sewers and lined and straight open channels. In low lying areas of Florida, ditches were also constructed to lower the groundwater table to improve the land for development. Although these systems solved local flooding problems, high peak flows and larger runoff volumes were generated. In addition, these systems were found to have little pollutant assimilative properties. An appreciation of impacts of stormwater on natural systems and receiving water bodies began to emerge in the early 1970's. Since then, regulatory legislation on the control of environmental impact from stormwater runoff has been evolving.

Stormwater management has been a topic of extensive study and debate over the last 20 years. Large amounts of funds were spent in the 1970s and 1980s researching the problem of non-point or diffuse pollution in stormwater runoff on both the national and site specific scales. These studies have made it possible to reasonably characterize where this pollution originates, how it migrates through the drainage network, and what technical solutions and management strategies can be used to control it. Unfortunately, much research in the stormwater area has been limited to the small projects. This is largely the result of funding agencies only wanting to focus on a single aspect of this problem and to support applied research in their narrowly targeted area.

After these findings and experiences, contemporary stormwater management is undergoing transition and re-evaluation. The enormity of the control solutions and regulatory enforcement is causing environmental managers to re-examine current policies.

Depending on the regulatory program, contemporary stormwater management includes efforts to control the magnitude and frequency of floods and to reduce the severity of water pollution events including erosion and sedimentation problems. They are implemented through standardized static designs which are based on idealized representations of the hydrology and pollutant characteristics. The operational procedures are fixed and are not related directly to the system dynamics. This approach is fostered by regulatory agencies who are faced with the need to standardize the designs for ease of enforcement since actual performance is not monitored. Alternative technologies or operating procedures are not readily accepted as they are generally required to include extensive monitoring programs.

Over the past 5 years, Florida's Indian River Lagoon has received national, regional, state, and local attention over its degradation and the citizen's action and multi-agency efforts to restore it. Degradation has included fish kills, the reduction of viable recreational and commercial fisheries, and loss of seagrass beds. Drainage practices on the watershed have been identified as the primary culprit in the slow demise of the Indian River Lagoon. Specific problems identified with the drainage are (1) the increase in the volume of freshwater runoff, (2) the increase in the deposition of organic sediments, (3) the reduction in water clarity due to the increased discharge of "colored" groundwater (a result of tannic acid) and suspended solids, and (4) eutrophication due to nutrient loadings. Poor flushing characteristics of lagoon segments intensify impacts due to runoff. Stricter stormwater regulations for new development and retrofitting of existing drainage systems within the Indian River Lagoon watershed are being considered by agencies as potential control and mitigation strategies. NASA's John F. Kennedy Space Center (KSC) comprises approximately 7% of the Indian River Lagoon's drainage basin and receiving waters surrounding KSC have been designated Outstanding Florida Waters by the State of

Florida. To aid NASA in meeting environmental commitments, predictive capabilities of impact on the watershed scale are being developed so that cost-effective mitigation strategies can be initiated.

The goal of this project is to take a first step in developing a watershed scale strategy towards managing drainage and urban runoff from KSC. This project focuses on analyses of data from the Industrial Area sub-catchment of KSC which discharges to the Banana River portion of the Indian River Lagoon. The principal objectives of this project are

(1) collect site specific data representative of typical KSC catchments for detailed analysis of viable options;

(2) develop a satisfactory methodology for estimating freshwater and pollutant loads from upland drainage areas;

(3) identify feasible control alternatives;

(4) determine the feasibility of retrofitting existing drainage networks from a multiobjective performance standpoint;

(5) determine whether a watershed control system will also satisfy new development requirements; and

(6) establish a water resources knowledge base for KSC.

Previous work in stormwater management, including information on the study area and the study approach, are discussed in Chapter II. Chapter III examines the water management objectives, the options available to KSC for stormwater management, environmental decision making, and documents the selection of channel retrofitting with wetland routing as the most feasible alternative. The methods for data collection and analysis are summarized in Chapter IV. Chapter V provides documentation of the calibration and verification of SWMM for use in hydrologic simulation of the study area. The runoff and total suspended solids loads for the existing land use and the maximum buildout development scenario are also presented. Chapter VI contains the results of the proposed watershed control system performance evaluation with respect to the multiple objectives. Chapter VII summarizes the findings and offers conclusions.

CHAPTER II

DESCRIPTION OF STUDY

Description of Study Area

The Industrial Area of KSC was selected as a representative catchment that is relatively developed and continues to be developed. It consists of 1900 acres of partially developed land set amongst undeveloped Florida Flatwoods on Merritt Island, Florida (a remnant barrier island). The study area, in relation to Merritt Island, is shown in Figure II-1. Approximately 700 acres are managed by the U.S. Fish and Wildlife Service (USFWS) as part of the Merritt Island National Wildlife Refuge (MINWR). Runoff from the catchment drains to Segment B2 (as identified by the Brevard County 208 study) of the Banana River (see Figure II-2). The study area catchment can be divided into six sub-catchments as delineated in Figure II-3. Land use for the entire study area and by individual catchment is summarized in Table II-1.

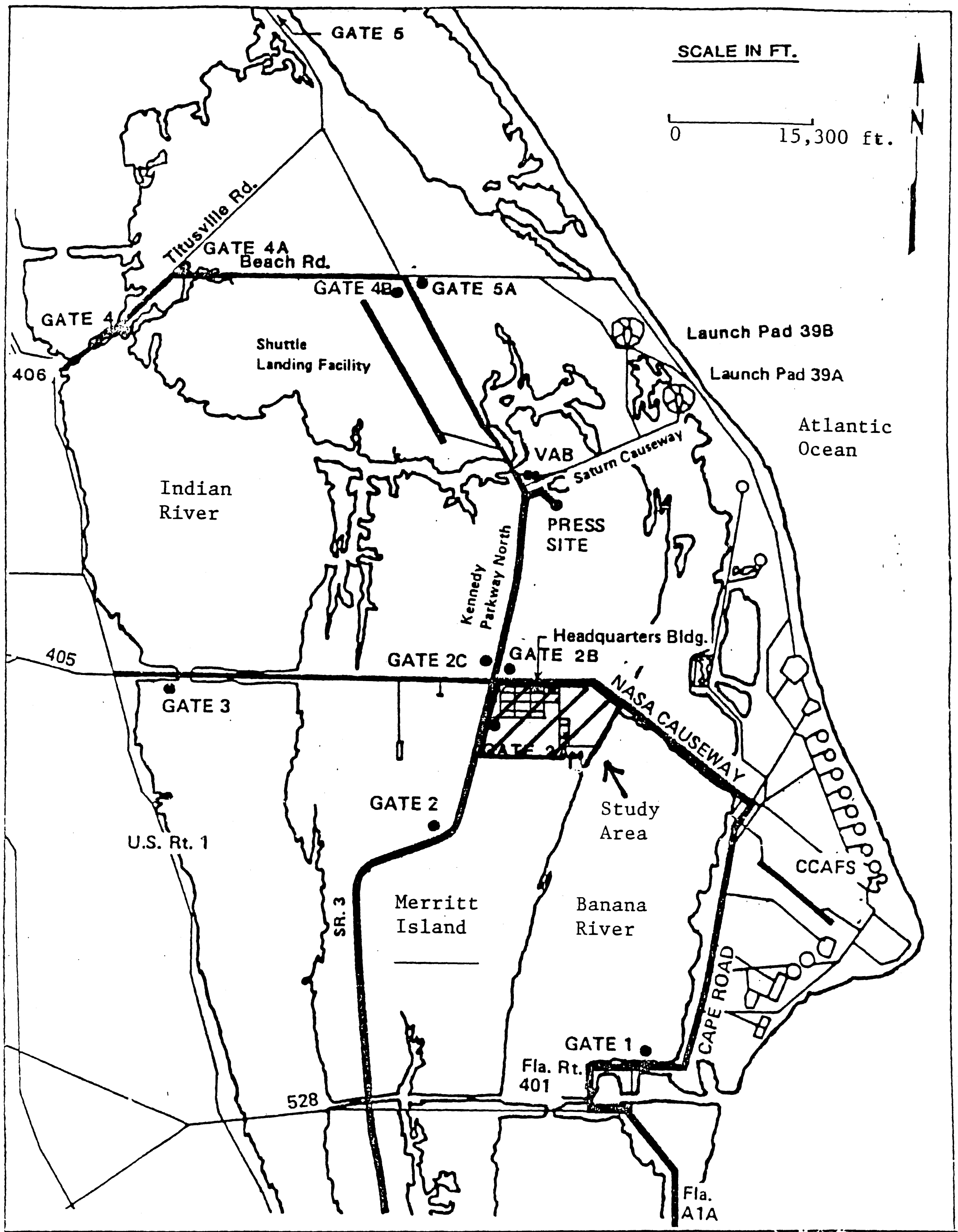


Figure II-1. Location of the Study Area on Merritt Island, Florida.
SOURCE: NASA, 1986.

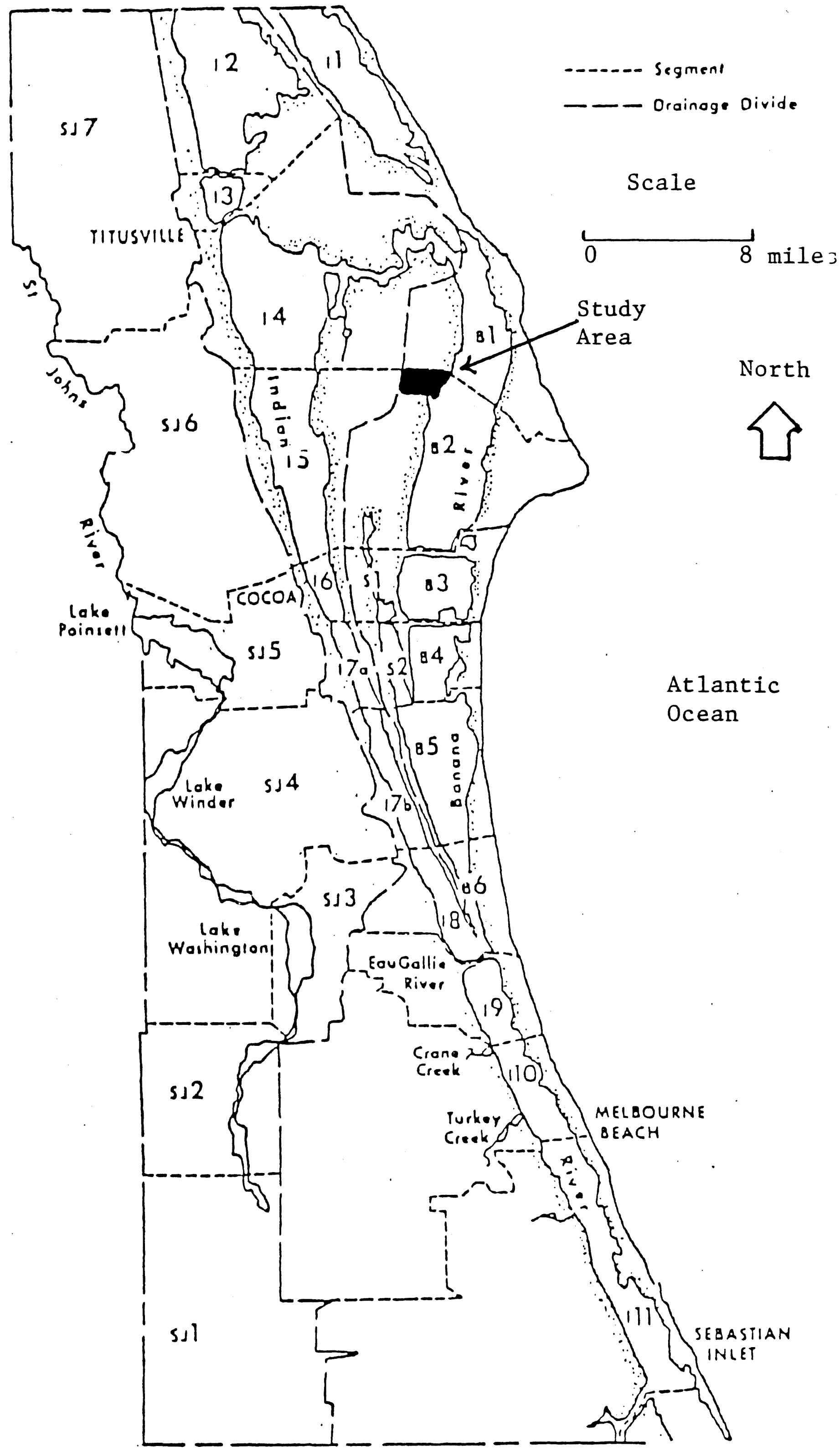
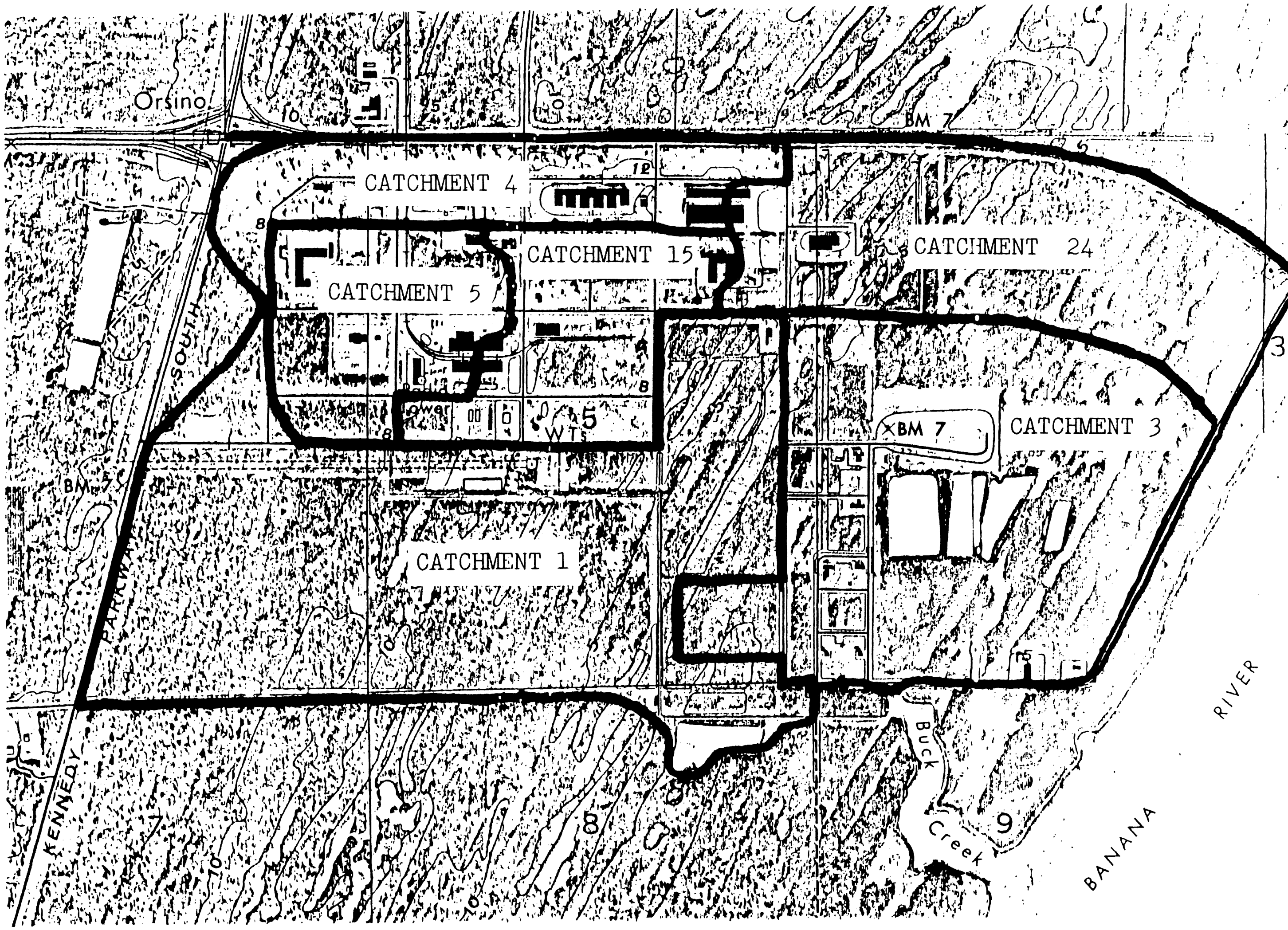


Figure II-2. Brevard County Water Quality Planning Segments.
SOURCE: Brevard County 208 Plan, 1979.



Scale: 0 2000 ft.

Figure II-3. Study Area Subcatchments. From USGS topo map, 1976.

Table II-1. Summary of Study Area Land Use at KSC.

LAND USE

CATCHMENT		Undeveloped Wetland	Undeveloped Upland	Ruderal	Open Channels & Borrow Pits	Impervious	Total
1	Acres	23.7	514.5	0.0	39.0	0.0	577.2
	%	4.1	89.1	0.0	6.8	0.0	100
3	Acres	95.0	233.1	64.2	34.4	63.1	489.8
	%	19.4	47.6	13.1	7.0	12.9	100
4	Acres	0.1	7.7	71.1	9.7	78.5	167.1
	%	0.1	4.6	42.5	5.8	47.0	100
5	Acres	0.0	33.1	38.5	0.0	61.2	132.8
	%	0.0	24.9	29.0	0.0	46.1	100
15	Acres	0.0	0.0	123.0	0.0	95.5	218.5
	%	0.0	0.0	56.3	0.0	43.7	100
24	Acres	46.8	179.2	26.0	19.0	44.6	315.6
	%	14.8	56.8	8.2	6.0	14.1	100
Total	Acres	165.6	934.5	284.3	102.1	281.7	1901
	%	8.7	49.2	15.0	5.4	14.8	100

Ruderal: Vegetation disturbed by man (e.g. lawn, roadsides).

Impervious: Roadways, rooftops, parking lots.

Geographical information for the study area was generated by updating existing soil and vegetation map files (Provancha et al. 1986) with cultural and urban features on the ERDAS geographical information system. The added features were digitized from KSC Master Planning maps at a scale of 1:9600. ERDAS utilizes a raster format; the pixel size for existing data files is 0.12 acres (73.79 ft. by 73.79 ft.).

As typical with most Florida Flatwood watersheds, the undeveloped portions of the study area can be characterized as having (1) extremely flat relief, (2) sandy soils, (3) dynamic shallow water table, and (4) scattered wetlands, locally referred to as interdunal swales. A very large portion of the relief for the area is the result of an extensive open channel drainage network constructed by the U.S. Army Corps of Engineers (COE) in the 1960's. The objective of the COE was to protect federal facilities from flooding by improving drainage. The drainage system has performed well (no flooding of facilities has been reported) and is continually maintained.

Buck Creek is a freshwater tributary to segment B2. Improved drainage channels have also been connected with Buck Creek. The remainder of the tributaries are man-made open drainage channels. None of these tributaries have been gauged for discharge in the past.

Segment B2 is approximately 16,000 acres in size, with a length of 7 miles and a width of 3 miles. The average depth is approximately 3.5 feet with the exception of barge channels which have a design depth of 12 feet. It drains approximately 31,000 acres of uplands from both Merritt Island and Cape Canaveral. Approximately 1800 acres are developed. The KSC Industrial Area accounts for 40% of this development.

Smith (1985) reports an astronomical tide elevation range of less than 0.03 feet in this part of the Indian River Lagoon. Changes in water level and current are due primarily to freshwater runoff and aeolian tides. Smith et al. (1987) used harmonic analyses of water level records near Melbourne, Florida to quantify components of the tide. They have shown that in the central part of the Indian River Lagoon nontidal variance characteristically accounts for 40-60% of the total. He notes that the non-tidal variation is quasi-periodic at best, but suggests that relative maxima occur approximately every five days as the result of a variety of forms of meteorological forces. Mixing and spreading of dissolved and suspended substances in the lagoon is a slow process; Carter and Okubo (1965) suggest a residence time of 150 days.

Yearly mean concentrations (1980-1985) of salinity, dissolved oxygen, total nitrogen, total phosphorous, and chlorophyll-a for Segment B2 are reported in Figure II-4.

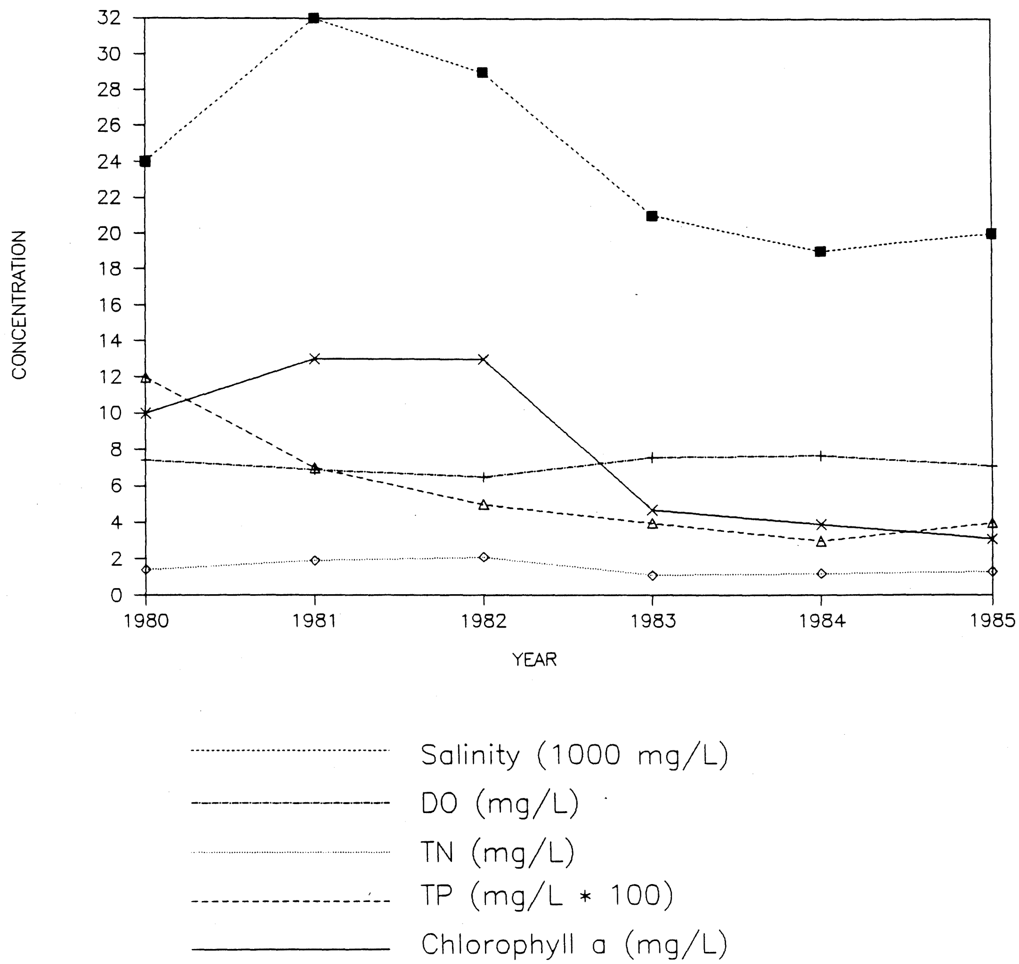


Figure II-4. Yearly Mean Concentration of Selected Water Quality Constituents in Segment B2 of the Banana River. SOURCE: Brevard County Department of Natural Resources (1987).

Peffer (1975) found that the bulk of the bottom sediments in the lagoon have a negative oxidation-reduction potential (Eh) and consequently concluded that the sediments act as a nutrient trap.

Four roles seagrass play in the ecology of an estuary are (1) habitat, (2) food source, (3) nutrient buffer, and (4) sediment trap. Among the major ecological values of seagrass meadows is the fact that many recreationally and commercially valuable fishery organisms utilize these systems for part or all of their life history (Fonseca et al. 1986). Seagrass and/or submerged aquatic vegetation covers approximately 70% of Segment B2's bottom (Provancha and Willard, 1986).

Receiving Water Impacts

The importance of the stormwater problem in terms of direct and indirect impacts on receiving waters is now receiving more attention. Heaney et al. (1979) stimulated interest in direct assessment of receiving water impacts by pointing out the very high cost of controlling runoff (an initial estimate of $200 to $400 billion for the nation). An early national inventory of available information on receiving water impacts (Heaney et al. 1981) revealed fundamental gaps in our knowledge. A more recent review, specific to estuarine systems (Odum and Hawley, 1986), notes that there are virtually no published studies which definitively show degradation of a significant estuarine area as a result of urban runoff. They go on to suggest that this is due to (1) the tendency for urban runoff and its effects to be interrelated with a host of other pollutant sources and their effects, and (2) a lack of recognition by estuarine scientists of the potential threat from urban runoff pollution.

The EPA's Nationwide Urban Runoff Program (NURP) sponsored several studies, including one in Tampa Bay, to measure wet-weather impacts on receiving waters. The Tampa Bay study revealed no acute toxicity to test organisms (Mote Marine Laboratory, 1984). In contrast, Odum and Hawley (1986) note that there is considerable circumstantial evidence pointing to urban runoff as a serious pollution source. They note examples along all coasts of the United States, including the Chesapeake Estuary and Biscayne Bay, in which damage from urban runoff is occurring but has not been well documented. In all cases the impact of urban runoff is masked by a variety of other pollutant inputs. Obviously, the assessment of impact is an extremely difficult task because (1) one must filter through numerous environmental variables over which the researcher has no control, (2) the definition of impact varies with the individual and their personal values, and (3) the impact is transient.

Perhaps the best synthesis of information on receiving water impacts and degradation of the Indian River Lagoon is found in Steward and Van Arman (1987). Although a great deal of data have been reviewed and synthesized, the conclusions of this report are largely ancedotal and lack data clearly documenting cause and effect relationships. Steward and Van Arman (1987) point out three major impacts on the Lagoon often associated with land drainage and runoff: (1) eutrophication due to nutrient loadings, (2) "muck" deposition, and (3) buildup of toxins and pathogens in biota. Poor flushing characteristics of lagoon segments intensify impacts due to runoff.

Turbidity is an expression of the optical quality of a water sample to scatter and absorb light. The penetration of light in water can be reduced by several factors including algal blooms, suspended solids, and "colored" water (tannic acid). Light penetration in estuaries regulates the productivity of phytoplankton and seagrasses (Rice et al. 1983; Heffernan and Gibson, 1983).

The roles of groundwater discharge in the lagoon ecology are poorly understood. Drainage of uplands around the lagoon has concentrated and increased flows from the surficial aquifer, routed them to stormwater outfalls, and altered the hydrology and constituent loads. Any benefits of natural diffuse groundwater flows have likely been lost. Thompson (1978) and Steward and Van Arman (1987) speculated that reduced light levels due to point source discharges of "colored" groundwaters were partially responsible for the poor conditions of the seagrass beds. Materials responsible for color in water exist primarily in colloidal suspension and are not due to dissolved materials (Black and Christman, 1963).

In a study on muck in the Indian River Lagoon, Trefry et al. (1987) found that the muck has been deposited over the last 20 to 30 years and that the two sources which contribute to the accumulation of muck are soil runoff and organic matter (e.g. plant remains). Using Turkey Creek as a field test site, Trefry et al. (1988) determined that iron-rich inorganic minerals (e.g. clays) make up 50-80% of the suspended sediment. Organic matter rich in nitrogen and phosphorous adds 10-40% to the suspended load.

In a comprehensive report to NASA, Laster (1975) reported that waters in the vicinity of KSC appear to be experiencing some degree of degradation. He speculated that nutrient materials derived from urban and agricultural runoff were the cause rather than effluents derived from "space oriented" activities. High concentrations of nutrients were found to be accumulating in sediments near the CCAFS sewage treatment plant outfall located 2.8 miles from the study area outfalls.

Management Perspective

In the 1970's emphasis on environmental management was based on risk aversion (strong anti-degradation philosophy). Due to the tremendous costs associated with the risk aversion approach, environmental risk assessment is swinging back to a cost-effective approach used in sanitary engineering prior to the 1970's. The cost-benefit-reliability approach to stormwater and drainage problems is by no means the only perspective, even today. During a 1984 workshop on contaminants in Florida's coastal zone (Delfino et al. 1984), 200 scientists concluded that they could not put the value of controlling contaminants in dollar terms. They recommended that a procedure should be established to accurately quantify ecological and esthetic values. To date, no accepted interdisciplinary procedure has been developed.

The goal of stormwater management is to reduce its effects to an acceptable level, a compromise between costs and benefits. It is impossible to predict costs and benefits precisely. Predicting the effects of the hydrologic changes (flow rates, water quality, sedimentation) is complex and much less accurate than say calculating peak runoff discharges. It is important not to lose perspective that the concern is not over the quantity of a specific chemical in water, but what effects the chemical at a specific concentration will have on biota and the subsequent benefit derived from the impacted water resource.

Program-related standards specifying pollutant concentrations and peak discharge ease implementation and enforcement of regulations. However, Heaney (1988) pointed out that this approach has been relatively ineffective given the performance results of dry-weather wastewater treatment plants under such an approach. Heaney noted that while it is relatively easy to run computer models to tabulate the statistics for a prescribed standard, it is an onerous effort to develop the actual field and laboratory information to support such a recommendation.

Increased environmental regulation has led to more pollution abatement measures. Increasing costs of pollution abatement services, construction, and monitoring have resulted in environmental management funds taking a larger percentage of available funds. Often this increased spending has led to little perceived benefit. For the case of stormwater regulation in Florida, performance standards have been promulgated even though performance is rarely monitored and the consequences of any violation are poorly understood or documented.

The nation continues to struggle with a national strategy for implementing stormwater programs to meet the objectives of the Clean Water Act. The Florida Department of Environmental Regulation (FDER) developed a regulatory program for the control of non-point sources. Livingston (1986) summarizes The Stormwater Rule as having established a performance standard for the treatment of stormwater. The performance standard is based on two properties of stormwater (1) annual storm frequency distribution, and (2) the first flush of pollutants. The rule's basic objective is to achieve 80-90% removal of stormwater pollutants before discharge to receiving waters. Thousands of stormwater basins have been designed and constructed to FDER performance standards; yet very few have been monitored for performance.

Livingston (1986) summarizes some of the major problem areas with the Stormwater Rule. A number of them apply to the KSC drainage network. The grandfathering of drainage systems constructed prior to the Rule is one. The retrofitting of existing systems that are causing water quality degradation is a major objective of the stormwater legislation. Little verification of removal effectiveness of systems permitted and constructed under the Rule is another. Recent performance evaluations of stormwater detention/retention basins meeting The Stormwater Rule specifications by Holler (1989) and Martin (1988), suggest that the State's goal is not being met. Because of poor retention system performance in flat, high water table areas like KSC, an effective alternative is sought. Wet detention basins are now being considered for general permit applications. Perhaps one of the most fundamental flaws identified is the promotion of a piecemeal approach to stormwater management which relies upon individual on-site stormwater management. A potential problem with this approach is the combined effects of individual randomly located detention basins which can increase downstream peak flow and cause channel scouring. The proliferation of numerous small on-site systems increases the difficulty of enforcing operation and maintenance requirements. The agency is now promoting a watershed management approach. In addition, the rule does not address other drainage associated problems such as sediment scour in drainage channels or the discharge of "colored" groundwater into the drainage system.

Methods/Approach

Field documentation of impact, especially long-term degradation, and effectiveness of control alternatives is difficult, laborious, and costly. Even when the studies are completed, the results are difficult to extrapolate to other development scenarios. Mathematical models offer a quicker and less costly approach to overall assessment of stormwater runoff and drainage problems. Models are efficient environmental quality management tools that are based on an accumulation of knowledge of the environmental phenomena to be managed; however, they are not intended to substitute for "real" data collection.

To minimize extensive and expensive field data collection, it was felt that runoff, water routing, and control can be reasonably predicted using existing models such as EPA's Storm Water Management Model (Huber and Dickinson, 1988). The requirements of the selected model were to help organize and visualize cause and effect relationships in the catchment and to assist in comparisons of control alternatives. To lend credibility to the predictions, local calibration/verification data were collected. For planning and overall assessment, continuous simulation (on the order of months or years) was used. Detailed simulation of selected events and synthetic design events were used for detailed performance analysis and preliminary design.

CHAPTER III

STORMWATER MANAGEMENT OPTIONS FOR KENNEDY SPACE CENTER

Multiobjective Strategy

As alluded to in the previous discussion, a number of objectives, some of them competing, were identified for a comprehensive stormwater management strategy. Stormwater is a complex problem that must be organized for analysis and decision making purposes. It is important to develop multiobjective water resources management that is rationally formulated to achieve the specified objectives. For this study, seven objectives were identified, as shown in the first column of Table III-1. Other objectives might be identified if such a study was applied the entire Indian River Lagoon watershed.

To aid in the identification of management alternatives, it was useful to identify some of the treatment/control principles that would be required of a system. Flood control can be achieved by maintaining hydraulically efficient drainage channels. Yousef et al. (1986) attempted to develop design criteria for

Table III-1. Multiobjective Analysis and Performance Measures for Comprehensive Stormwater Management Strategy.

OBJECTIVE	PERFORMANCE MEASURE
Protect facilities from Standard Project Flood	Prevent road flooding at 3 selected locations
Maintain aerobic sediments in channel	Minimize water depth (not to exceed 6 ft.)
Reduce "colored" groundwater discharge	Predicted annual groundwater discharge
Maintain estuarine salinity	No. of days in excess of 5 cfs. or cumulative discharge
Reduce pollutant and sediment loading to the estuary	Event mean concentration of total suspended solids (DER goal = 80% reduction)
Prevent mosquito infestation	Minimum water depth (0.5 ft.), number of days with no flow
Enhance channel littoral zone as natural habitat and treatment system	Annual water level frequency

retention/detention basins based on nutrient removal and transformation between the overlying water and bottom sediments. They found that by maintaining an aerobic environment at the sediment-water interface and in the upper sediment layer, nutrient removal was enhanced. They recommended the design of shallow basins no greater than 5 to 6.5 feet deep to maintain an aerobic environment. Reduction of "colored" groundwater discharge could be accomplished by raising channel invert elevations several feet or by storing water in the channels to reduce the groundwater hydraulic gradient to the channel. Black and Christman (1963) found that prolonged storage has only a slight effect on the color value of water. Contemporary water treatment plants use alum coagulation with adequate pretreatment for the removal of color from water supplies. The reduction of freshwater discharge can be accomplished by reducing the imperviousness of the catchment, runoff retention, water reuse, or by enhancing evapotranspiration. Sediment loads and pollutants associated with particulates can be controlled using sedimentation and/or filtration. This is usually accomplished by runoff detention or retention. Stowell et al. (1985) suggested that the best approach to controlling mosquito production in natural wastewater treatment systems is to design the system so that natural predators of mosquito larvae (e.g. mosquitofish, dragonfly, and a variety of water beetles) will thrive throughout the system. These predators are strict aerobes;

therefore, they recommended that no part of the system be hydraulically static and that shallow areas not be allowed to form. Fluctuating water levels are usually identified with maintaining littoral zone vegetation species diversity.

Measures of performance that relate system capabilities to the objectives were then generated. The performance measures were based on regulatory guidelines and literature suggestions. The next step involved generating alternative schemes for attaining desired objectives. For this study, nine alternatives were identified (Table III-2).

Selecting Management Options

Environmental policy is established in the private sector, but relies heavily on technical and social steering committees. When addressing water pollution control or water resources management, the regulatory agency must somehow identify design or performance standard alternatives. The traditional engineering approach has been to use a single criterion of economic efficiency. This made it possible to use cost-benefit analysis as a basis for ranking alternatives. However, this method can not handle other objectives such as environmental quality or social well being (Heaney, 1979a). Therefore alternative procedures were sought.

Table III-2. Alternatives to Implement a Watershed Stormwater Management Strategy.

MANAGEMENT OPTIONS:

1: Retrofit canals with a weir.

2: Retrofit canal w/ weir and wetland routing.

3: Construct individual retention basins for old & new facilities.

4: Retrofit canals and construct individual basins.

5: Construct central off-line wet detention area.

6: Construct off-line wet detention & retrofit canals.

7: Construct storage reservoir and use for irrigation.

8: Construct storage basin, combine w/ wastewater effluent, and use as cooling water makeup.

9: Take no action.

A large number of management strategies have been developed over the years. The feasibility of each is based on site specific criteria. Each and every alternative could be evaluated; however, common sense dictates that resources could be better spent by a detailed evaluation of those most likely feasible. Therefore preliminary screening techniques are very useful in eliminating options from further analysis. Many technical and social factors come into play for such environmental decision making, including judgment and experience. Social choice analysis of water resources and environmental problems is a formalized area of study of its own. Straffin (1979) provides an informative synopsis of social choice theory with respect to environmental decision making. These formalized techniques were reviewed and an appropriate analysis method synthesized from the theory as a formalized procedure to aid in the selection of a feasible watershed management approach.

The procedure must account for technical fact, experience, multiple objectives, pre-emptive goals, uncertainty, social preference, and decision-maker bias or favoritism. The new age of microcomputers has led to the development of decision support systems that utilize data and models to aid environmental managers. However few specific computational procedures for examining the social choice questions have been applied. Therefore a simple matrix based system was used for this study. Some of the criteria, considerations, and assumptions incorporated were:

- Decision making based on intensities of preference (Cardinal Utility).
- Because the benefits of such a water quality system will be difficult to measure or assign a value to let alone an immediate realization of benefit, cost must carry a high weight.
- With the huge public investment in Federal facilities that support the national space exploration and industry mission, flood protection must be a pre-emptive objective (satisfied first before others).

The results of this analysis are found in Table III-3. It can be seen that the option of channel retrofitting with wetland routing will likely provide the best overall system performance. However, its performance will unlikely dominate other options for every objective. Option specific objectives and performance measures were then developed for detailed analysis of the selected option (see Chapter VI, Table VI-1).

Table III-3. Decision Aid Matrix for Selecting Feasible Management Strategy for Detailed Analysis.

OBJECTIVE	WEIGHTS	OPTION 1	2	3	4	5	6	7	8	9
Flood Control	5	10	10	10	10	10	10	10	10	10
Reduce TSS Loading	0.8	6	9	6	8	6	8	9	9	4
Reduce Groundwater Discharge	0.8	8	8	1	8	4	8	8	8	1
Reduce Freshwater Discharge	0.8	4	8	5	8	3	8	9	9	2
Maintain Diverse Vegetation	0.5	5	5	5	5	5	5	5	5	5
Maintain Aerobic System	0.8	3	7	8	8	5	2	5	5	8
Low Operation & Maintenance	0.8	9	8	4	3	8	6	2	1	7
Low Capital Cost	1.0	9	8	2	1	4	1	2	1	10
Proven Technology	0.5	5	8	6	6	9	8	9	5	3
Does Not Promote Mosquitoes	0.6	7	5	4	3	5	4	5	5	7
WEIGHTED SCORE		94.8	102.8	82.8	90.0	89.2	90.0	93.2	88.4	87.6

<u>Note:</u> The key to the option numbers is found in Table III-2, page 27.

CHAPTER IV

DATA COLLECTION AND SUMMATION

There was a real lack of extended, reliable databases on streamflow, lagoon water level, and water quality for the Indian Lagoon. Available data were of short time series (e.g. a year) and were generally collected with a narrow set of objectives in mind. In addition, data in electronic form are not located in one centralized location. Therefore data collection stations for this study had to be established. Although this contributed to the accumulation of project specific data bases, it was required to add credibility to the results of this project.

Precipitation

Analog recordings of precipitation were made by a Belfort rain gauge at the CIF Antenna site which is located approximately 1.1 miles north of the study area. The instrument is included in, and therefore meets the standards of the National Atmospheric Deposition Program (NADP). The analog charts were evaluated by "hand" and hourly rainfall intensities extrapolated.

Work by Dreschel et al. (1988) that estimated chemical loadings to the Indian River from atmospheric precipitation was reviewed. However, this initial investigation ignored atmospheric loading. The surrogate parameter (i.e. in substitute of all other pollutant constituents) used in this study was Total Suspended Solids (TSS). Other chemical loadings were assumed to be represented as a fraction of the TSS loads. Dreschel et al. (1988) do not present data on particulate loadings. Therefore, it was assumed that atmospheric precipitation does not contribute to TSS directly.

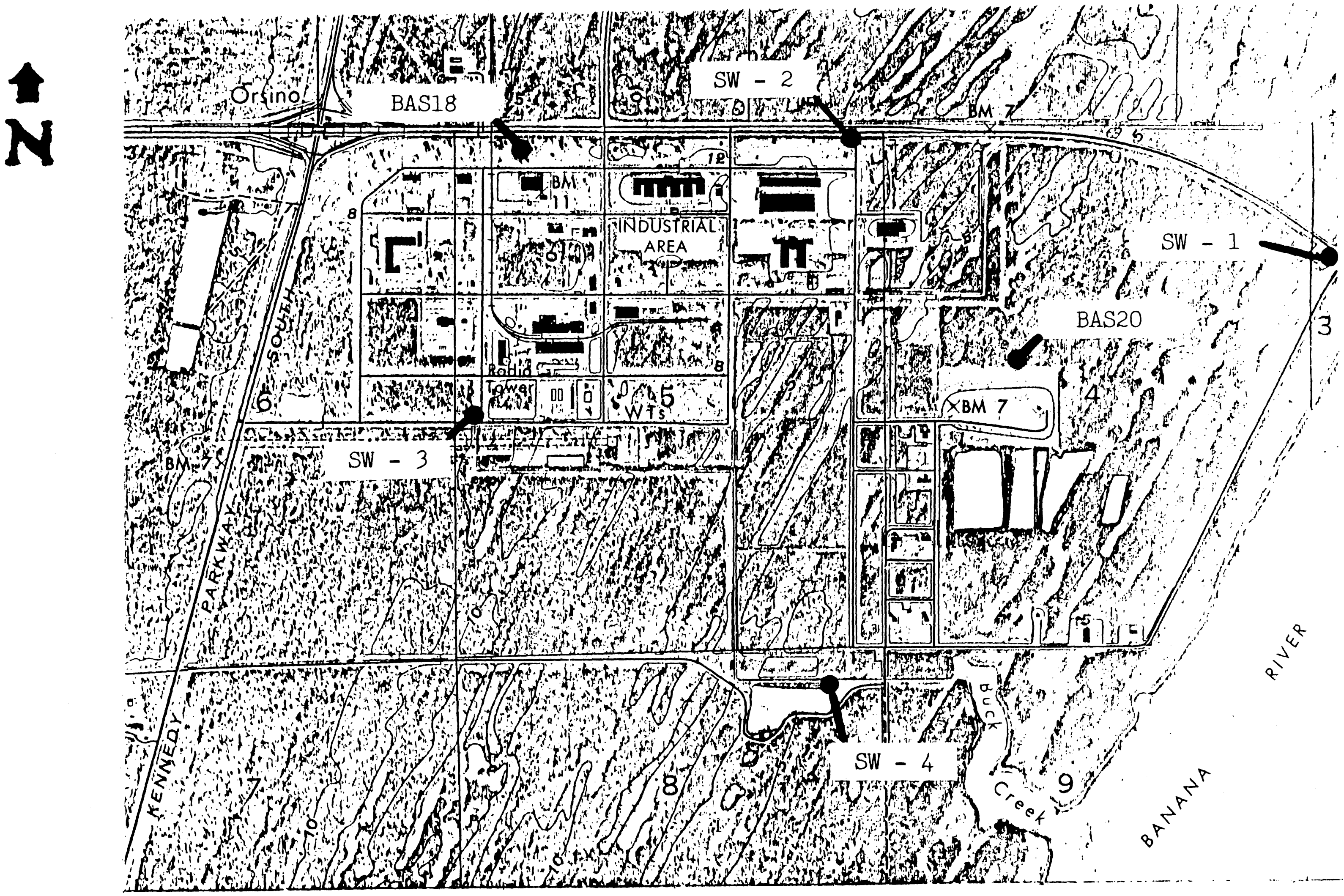
The major disadvantages to running continuous simulation hydrologic models are the computational time and computer hardware requirements. Therefore, a single year was selected to represent typical conditions. Continuous hydrologic simulation will be driven with the 1951 hourly precipitation record for a gauge in Melbourne, Florida. This particular year was selected because, on the basis of synoptic statistics of intensity, duration, and volume; it was deemed "typical" for Melbourne (Dwornik, 1984). Goforth (1981) demonstrated that a "typical" year will result in production functions very similar to those developed from much longer periods of record.

Stage-Discharge Relationship

There are two major outfalls to the Banana River from the Industrial Area catchment. All surface discharges, with the exception of Catchment 3 ("Hot Fire Area"), were gauged to continuously record channel water levels. Gauging stations were located as a function of acceptable channel characteristics and availability of existing data collection equipment. The locations of gauging stations are shown in Figure IV-1. A summary of stage data collected for this study is found in Appendix A.

Each gauging station consisted of a continuous analog water level recorder and a permanent mean sea level reference marker. Station SW-1 and SW-4 used Stevens Type F Water Level Recorders with multi-speed timer and stations SW-2 and SW-3 utilized a WEATHERtronics Model 6530 recorder with an 8 day spring wound clock.

Original study plans called for the development of stage-discharge curves for each gauging station. However, Stations SW-1 and SW-4, located at outfalls near the Banana River, were significantly influenced by aeolian tides (wind setup) of the lagoon. These quasi-periodic tides cause reversible flow in the channels near the outfalls making it impossible to develop rating curves for these stations. Discharges at these stations were estimated by simulation. These computations are addressed in the hydrologic simulation section.



Scale: 0 2000 ft.

Figure IV-1. Location of Data Collection Stations. From USGS topo map, 1976.

In order to develop rating curves for the two gauged upland sub-catchments, discharge was measured using two methods. One method was the velocity times area method (U.S. Geological Survey, 1980) where velocity was measured using a QUALIMETRICS Model 6665 Water Current Meter. For extremely low flows, flow was measured directly using a graduated cylinder and a hand-fashioned flume. Because (1) peak storm stages last on the order of an hour, and (2) the remoteness of the study area, an insufficient number of direct peak (high stage) discharge measurements were made to develop a regression equation from the log-log transformation of stage and discharge data. In other words, to represent the entire range of stage data, the regression equation would have to be extrapolated to higher stages where no direct discharge measurements were made. Because the log-log transformation technique is dependent upon sufficient empirical data rather than physics, such an empirical relationship could lead to highly inaccurate estimates of discharge at high stages.

The Manning equation is widely used for open channel design with uniform flow and can be manipulated to represent a stage discharge rating curve. Due to the extreme width of the channel versus the shallow depth, and the high friction factor of the bed, computations show that flow in these channels is turbulent and rough (L-Range) when the depth is less than approximately 1.8 feet. Theoretically, the Manning's equation is not valid in this flow range.

From the Manning's equation, Christensen (1985) provides power formulas to describe various types of flow and selected channel geometries. The L-Range Power Formula for describing flow in a wide, trapezoidal channel under a normal flow regime is as follows:

$$(Q/(L * \mathrm{sqrt}(S_b)) = (A_o\text{^}1.6667/Po\text{^}0.6667) \qquad (1)$$

where Q = discharge (m^3/sec),
L = roughness coefficient (m^0.5/sec),
S_b = bed slope (m/m),
A_o = area of the channel (m^2),
P_o = wetted perimeter of the channel (m),

and

$$L = (6.46 * \mathrm{sqrt}(g))/k\text{^}0.3333 \qquad (2)$$

where g = acceleration due to gravity (m/sec^2), and
k = equivalent sand roughness.

Solving for Q and inserting the geometry for a trapezoid yields,

$$Q = \left[\frac{(d_o * (1 + S*(d_o/b)(6.46*b* @\mathrm{sqrt}(g*S_b))\text{^}0.6)}{(1+2*(d_o/b)*@\mathrm{sqrt}(1+S\text{^}2))\text{^}0.4 * (K\text{^}0.3333)\text{^}0.6}\right]\text{^}1.6667 \qquad (3)$$

where d_o = water depth (m)

and

for Station SW-3,
S = side slope = 1/2 = 0.5
b = bed width = 4.57 m (15 ft)
S_b = bed slope = 0.25/1000
k = equivalent sand roughness,

where k = [n*25]^6 = 0.1143; where n = 0.027

for Station SW-2,
$S = 1/2$
$b = 4.57$ m
$S_b = 1.0/1600$
$k = 2.140$ where $n = 0.044$

Flow in the gauged channels was found to be represented by a normal depth, wide trapezoid channel. Manning's n was determined for each gauged channel through iterative trial and error solutions for n with a known water depth. For SW-3 and SW-2, Manning's n was found to be 0.027 and 0.044, respectively.

A conversion factor is used to convert from metric to the more traditional English units. Recorded stages at each station are shown in Figures IV-2 and IV-3; the respective rating curves are shown in Figures IV-4 and IV-5, respectively. Divergence between the observed and theoretical discharges are thought to be related to the Manning's n term. Observations in vegetated channels by Christensen (1976), Petryk and Bosmajian (1975), and Turner et al. (1978) indicate that Manning's n varies and should not only be a function of the type and degree of vegetation but also of the flow depth as well. Resistance to flow is greater for emergent vegetation conditions which are common in the study area during dry-weather periods. Theoretically it is incorrect to assume a constant n value to extrapolate the rating curve for vegetated channels as was done in this

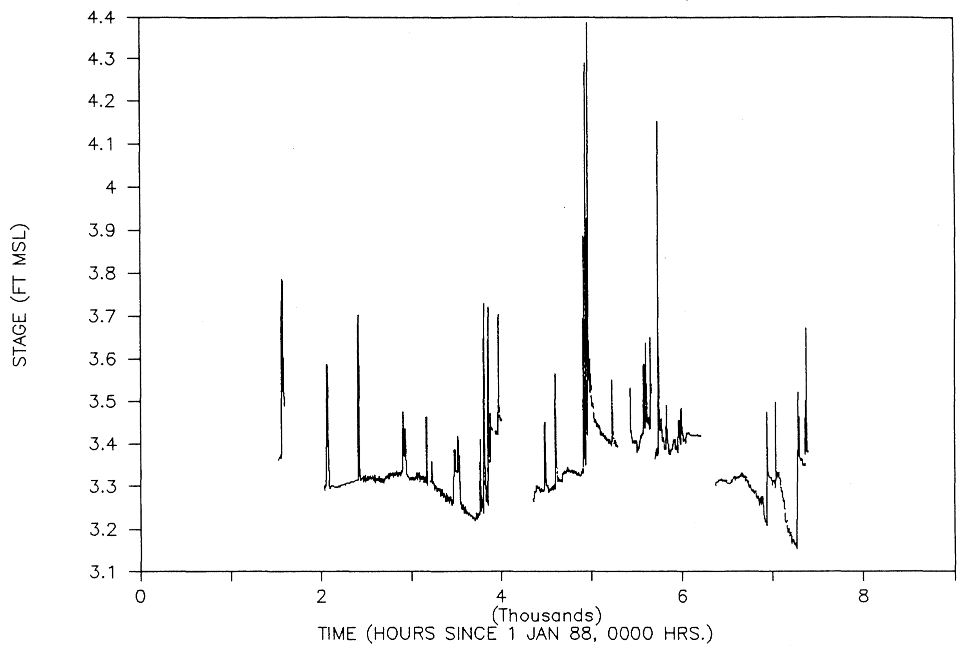


Figure IV-2. Stage Data Collected at Station SW3 During the Study Period. (Invert Elevation = 3.35 ft. MSL)

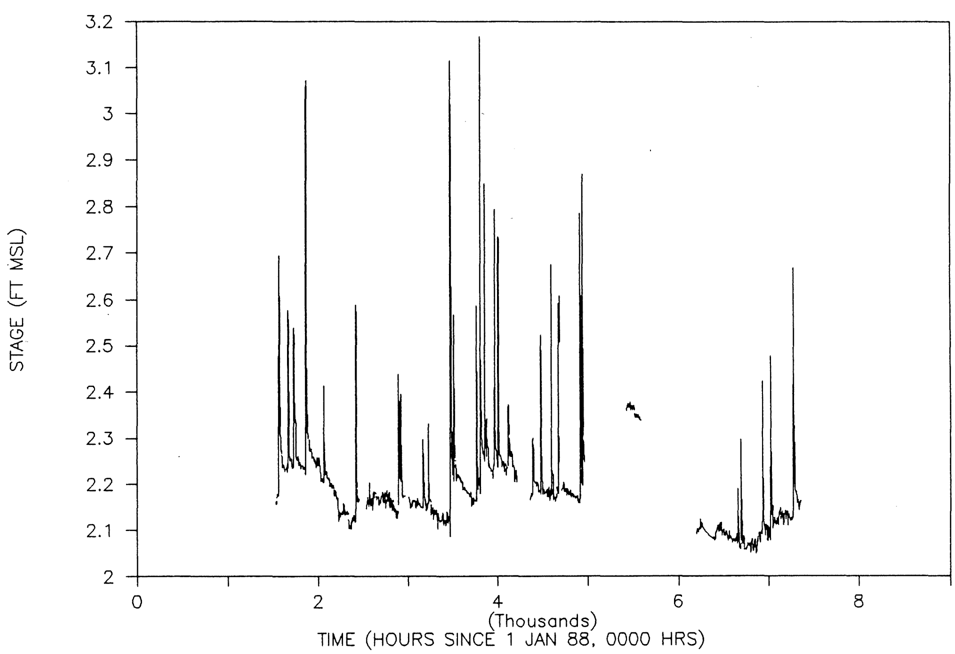


Figure IV-3. Stage Data Collected at Station SW2 During the Study Period. (Invert Elevation = +2.0 ft. MSL)

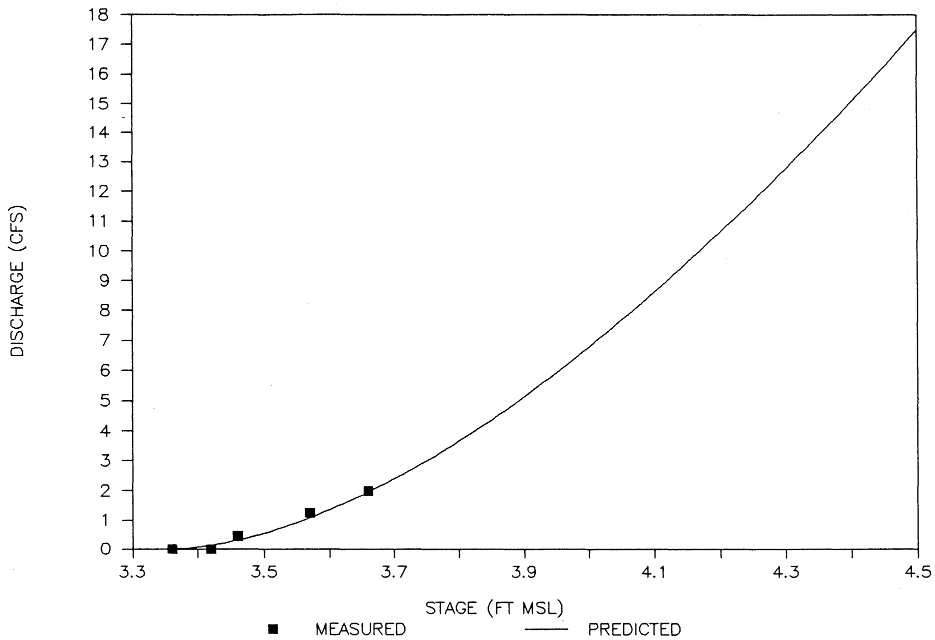


Figure IV-4. Stage-Discharge Rating Curve for Station SW3.

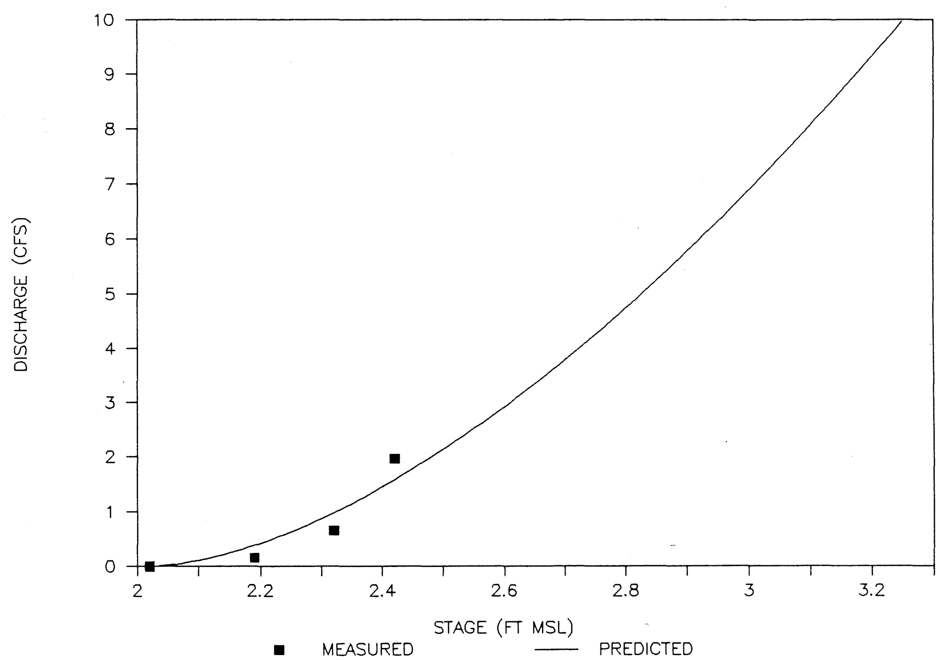


Figure IV-5. Stage-Discharge Rating Curve for Station SW2.

study. This likely accounts for some of the variability between the observed and predicted discharges. However for high flows (a concern to this study) the Manning's n approaches a constant.

Groundwater Levels

Because not enough continuous water level recorders were available, water table elevations in the study catchment were measured manually. KSC observation wells BAS18 and BAS20, both located in the Industrial Area catchment (see Figure IV-1, p. 34) and in the surficial aquifer, were used to measure the shallow groundwater table elevation. Measurements were taken on a weekly or twice weekly frequency. Groundwater hydrographs for these stations are plotted in Figure IV-6.

Water Quality

Generally, the concentration and annual load to receiving water bodies from urban runoff is comparable with that from secondarily treated domestic wastewater (USEPA, 1983). However, substantial variation exists from site to site and seasonally at a site.

Jones (1986) collected data on stormwater runoff quality in the VAB area of Launch Complex 39. The results, summarized in Table IV-1, are based on composite samples

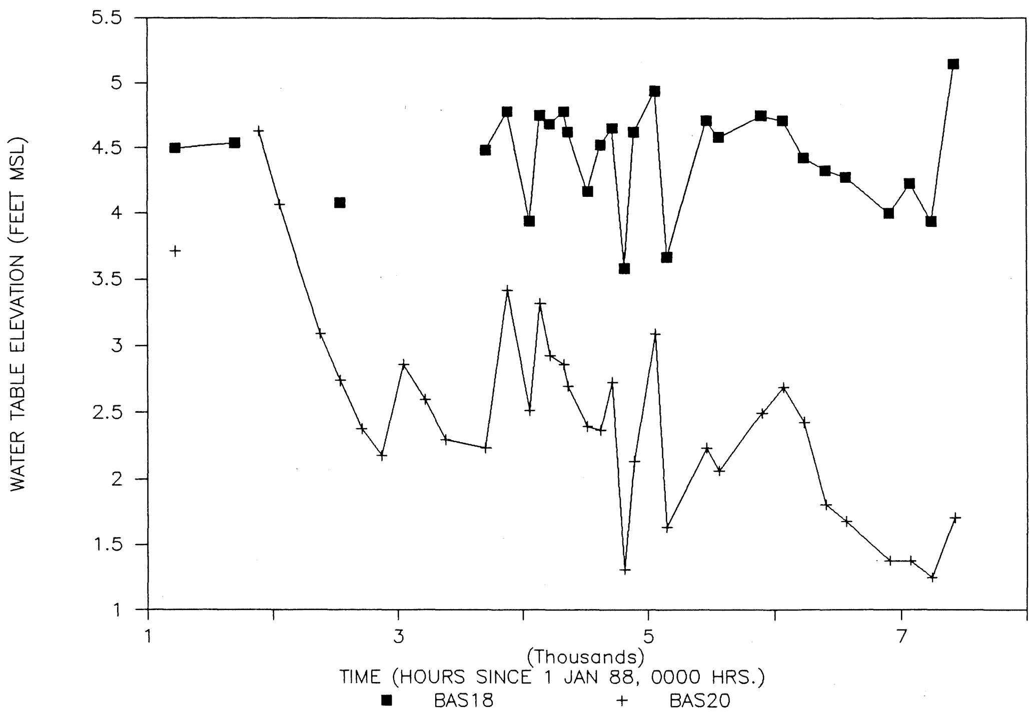


Figure IV-6. Groundwater Hydrographs for Observation Wells BAS18 and BAS20.

Table IV-1. Stormwater Quality for the VAB Area of KSC for 1986.

Range of Composite Runoff Quality at Discharge Sites During Three Rain Events.

Parameter*	Site 1	Site 2	Site 3
Cl	124-187	30-97	10-72
*pH	7.5-8.4	7.2-7.8	7.0-7.3
*Conductivity	730-1298	415-640	133-640
BOD	5-7	4-353	4-9
*Turbidity	2-50	12-68	1-10
COD	63-71	50-591	34-70
Hardness	232-280	164-238	49-102
Alkalinity	141-184	145-254	34-46
TDS	589-770	224-382	96-216
TOC	16-20	14-16	16-24
TKN	0.8-2.9	0.6-4.3	0.7-1.4
NO2	<0.01-0.01	<0.01-0.01	<0.01
NO3	0.20-0.50	<0.02-1.80	0.30-1.30
NH4	0.13-0.16	<0.10-0.65	<0.10-0.61
Ortho PO4	0.10-0.13	0.30-0.60	0.03-0.12
Total PO4	0.29-0.37	0.71-1.16	0.09-0.24
Oil & Grease	<0.2-4.4	<0.2-4.0	<0.2- 1.5
TSS	19-73	52-398	4-27

*Reported in mg/L, except pH and turbidity units and conductivity (umhos/cm)

Source: Jones, 1986.

from three rain events in 1986. They are comparable with those generated by the Nationwide Urban Runoff Program (NURP) (USEPA, 1983) from commercial areas. Site 1 of the Jones' study area is apparently influenced by backwater from the estuary. Ryan and Goetzfried (1988) report monthly water quality results in canals within the study area tributary to Buck Creek.

Total Suspended Solids

It was beyond the scope of this project to perform a comprehensive water quality sampling and analysis program. However to support predictions of performance, it was decided to monitor for a surrogate parameter to represent stormwater/ drainage runoff quality. The NURP (USEPA, 1983) found that TSS is an acceptable surrogate parameter for determining pollutant loadings and effluent treatability (sedimentation as primary treatment). Whipple and Hunter (1981), Yousef et al. (1986), Martin (1988) and Ferrara and Witkowski (1983) have found that, with the exception of perhaps nitrogen, settling of suspended sediments is fairly well correlated with pollutant removal because of their affinity for suspended sediments via the sorption process. The sources of suspended solids include watershed washoff processes and conveyance channel scour.

Flow weighted composite samples were collected for storm events. Samples were collected by an ISCO Model 2700

Wastewater Sampler with a change in water level actuator. A positive change in water surface elevation of 0.08 ft. was the threshold for triggering the sampler. A one liter sample was collected immediately with 23 successive one liter samples collected every 30 minutes. A composite storm sample based on relative discharge through the control channel was prepared from these samples in the laboratory. Anytime the water level recedes below the actuator, sampling was discontinued. Because only one sampler was available (and was on call) its location was rotated between the sub-catchments. Samples were analyzed for Event Mean Concentration (EMC) TSS using Standard Method 209C (Standard Methods, 1985). The EMC's of TSS for 11 events are shown in Table IV-2.

Settleability of Runoff Pollutant Loadings

Sedimentation is the removal of solid particles from suspension by gravity (Viessman and Hammer, 1985). It is a commonly used primary treatment process in the water and wastewater industry as well as in natural water bodies where the near static pools occur (e.g. floodplains and wetlands) where sufficient time is allowed for gravity settling. Depending on the concentration and the tendency of the particles to interact, theoretically four classifications of settling can be described, (1) discrete particle, (2) flocculant, (3) hindered, and (4) compression. Settling of

Table IV-2. Total Suspended Solids Loadings in Runoff From the Study Area.

Station: SW1
Catchment: 2

SAMPLE	DATE	HOUR	RAINFALL (Inches)	RUNOFF VOLUME (ft.^3)	TSS EVENT MEAN CONC. (mg/L)
2SMP1	21Feb88	1512	0.35	No Data	337
3SMP1	28Feb88	0100	*NOTE*	No Data	25

Station: SW2
Catchment: 4

SAMPLE	DATE	HOUR	RAINFALL (Inches)	RUNOFF VOLUME (ft.^3)	TSS EVENT MEAN CONC. (mg/L)
1SMP2	5Nov88	1300	0.44	55051	583

Station: SW3
Catchment: 5

SAMPLE	DATE	HOUR	RAINFALL (Inches)	RUNOF VOLUME (ft.^3	TSS EVENT MEAN CONC. (mg/L)
1SMP3	19Mar88	1845	1.65	99515	220
2SMP3	9Apr88	1330	NO DATA	42047	310
3SMP3	11May88	1900	0.3	3497	135
4SMP3	14Jun88	0500	0.04	34819	210
5SMP3	13Jul88	1900	0.25	No Data	283
6SMP3	23Jul88	1500	1.05	65881	1483
7SMP3	25Jul88	1400	1.35	222234	172
10SMP3	5Nov88	1300	0.44	25437	460

<u>Note:</u> Sampler activated by wind setup.

stormwater loads in Florida can be generally described by discrete particle settling (coarse material) or flocculant (e.g. muck).

The settling velocity of a particle can be calculated for ideal situations (e.g. spherical particles) as a function of particle density, particle size, viscosity of the settling medium, and the density of water using Stoke's Law. Unfortunately, particulate loads in urban runoff are not of idealized shape, and the determination of particle size and density is very laborious. A more pragmatic approach is to estimate the distribution of settling velocities empirically using standard settling column tests.

Testing runoff by the settling column procedure is a very useful and relatively inexpensive technique. It provides important information about the characteristics of runoff that are useful in a general sense as well as having direct application for the evaluation of detention basins where sedimentation is the removal mechanism. Grizzard et al. (1986) showed comparable results between laboratory studies of quiescent settling of stormwater and field performance data for full-scale detention ponds. In a similar type comparison, Martin (1988) found the order of constituent removal was about the same; however, the size of the reductions was about 10 to 50% less in the detention pond than in the laboratory.

Settleability tests were conducted in quiescent settling columns made of Pyrex glass, 0.17 feet ID and 1.5 feet in depth. The settling column depth was determined by available columns. Although they would be considered shallow from the nationwide perspective, they are considered fairly representative of basins on Merritt Island where retention basin depth is very limited by the extremely shallow depth to water table. Rather than sampling ports, graduated pipettes were used for sampling TSS at the 1.2 foot depth as a function of time. A total of eight settling tests from three sites were used to provide an overall picture of the settleability of the runoff solids at KSC. As was expected and observed in many stormwater studies, a large percentage of the solids was removed within several hours (e.g. 50% in two hours).

Reaction kinetics are often used to describe environmental processes. They are commonly used to describe the settling of particles where the settling process is considered a "reaction"; such generalities are particularly useful in system performance evaluation applications. Several integer kinetic orders were tested using regression techniques on transformed and untransformed data. Concentration data regressed were averaged concentration values for the respective time. Both first and second order reaction equations produced good coefficients of determination (R^2 = 0.92 and 0.93, respectively). However, using the coefficients generated from these techniques, the resulting curve fit provided a poor visual fit to the scattered raw data. As an alternative, the differential method procedure for determining the reaction order for isothermal irreversible reactions in a perfectly mixed, constant volume reactor (Levenspiel, 1972) was used to determine if a non-integer order kinetics applied to the settling of the effluent. This procedure relates the rate of reaction and the concentration of reactant by:

$$-dC/dt = r = kC^n \tag{4}$$

where r = reaction rate,
k = rate constant,
C = concentration of reactant, and
n = reaction order.

The reaction order is found by plotting reaction rate, dC/dt, versus concentration, C, where the slope of the best fitting line represents the reaction order. As Figure IV-7 illustrates, no best fit line could reasonably be applied.

Therefore after consulting work by Nix (1982) who noted that TSS settleability is fairly represented by first-order kinetics, it was decided to use the simple first-order removal equation:

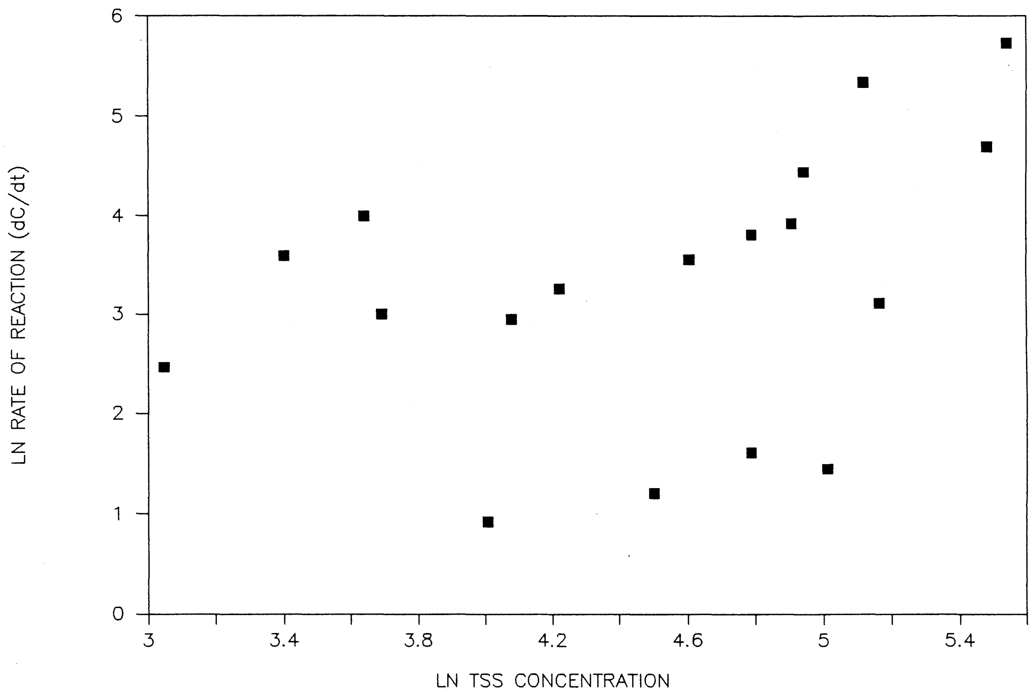


Figure IV-7. Results From the Differential Method for Reaction Order Determination as Applied to TSS Settleability in KSC Stormwater.

$$C/Co = 1 - (Rmax * (1 - EXP\hat{}(-kt))) \qquad (5)$$

where Co = initial concentration (mg/L)

Rmax = maximum removal fraction

k = rate constant (1/hr.)

t = detention time (hrs.)

Rmax and k were adjusted by trial and error until a reasonable fit of the scatter plot was obtained. A "reasonable fit" was based on experience and visual inspection. The selected coefficients are:

Rmax = 0.85 (Nix (1982); Whipple and Hunter (1981) suggested 0.75 for TSS).

k = 0.43 (Nix (1982) suggested that 0.108 was reasonable for stormwater effluent).

The results of this trial and error fit to the raw data are shown in Figure IV-8. The raw settleability data are shown in Table IV-3.

Rating Curve for TSS Load

Because of the difficulty in predicting TSS loads based on the physics of buildup-washoff processes, loads are commonly predicted using empirical equations such as rating curves. Field measurements on stream and channels often warrant application of a simple empirical relationship between suspended load and discharge, of the form:

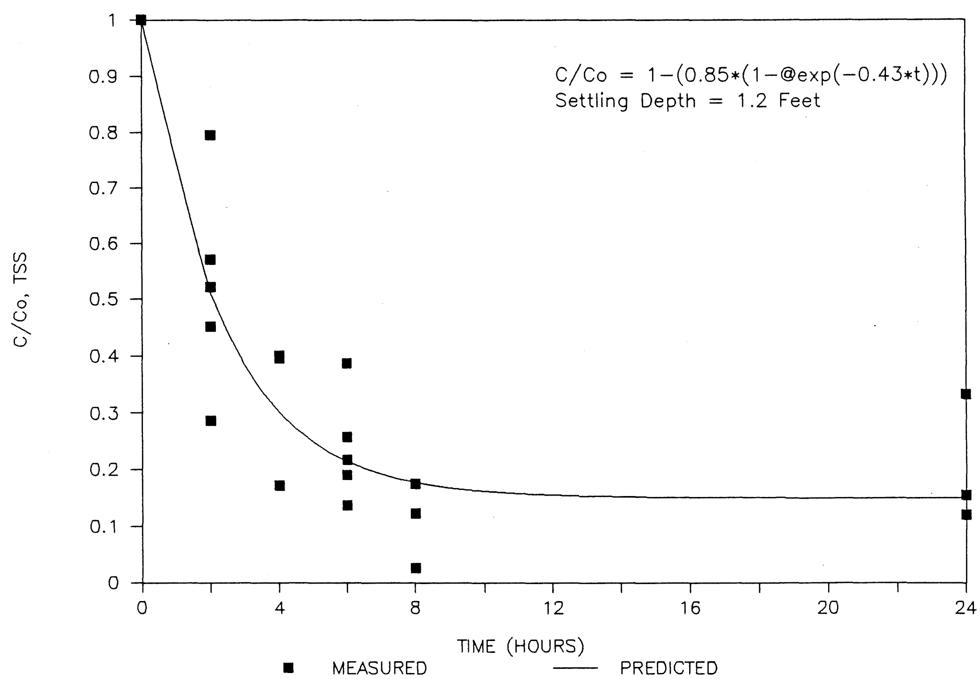


Figure IV-8. Results of Settleability Tests for Total Suspended Solids Load in KSC Stormwater Effluent.

Table IV-3. Raw TSS Settleability Data.

Time (hrs.)	TSS Concentration (mg/L) Sample: 7SMP3	2SMP3	2SMP1	1SMP2	1SMP3	4SMP3	10SMP3	6SMP3
0	172	310	337	583	220	210	460	1483
2		140		167	175	120	240	
4	68		135					
6		120		150	30	40	100	255
8	21		59					
24				90		70	55	

$$M = a * V^b \tag{6}$$

where M = total load (mass)

V = runoff volume, and

a,b = regression coefficients.

The results of the simple regression analysis on the transformed data are presented in Figure IV-9. The prediction of the rating model versus actual measured data is shown in Figure IV-10. Note the general increase for these two locations in the variance of loads as runoff volume increases. A number of field studies have shown loading data as scattered as these (Diniz and Espey, 1979; Smolenyak, 1979). For example, after using a log-log transformation of TSS data for 260 events, Smolenyak (1979) reported a coefficient of determination of 0.56 from regression analysis.

Channel characteristics at KSC undergo continual evolution (e.g. from scraping to various densities and heights of vegetation) over the year. It is hypothesized that this accounts for much of the variability. More data over a number of channel cleaning and revegetation cycles would likely produce a better rating curve. However for analysis, this relationship provides general runoff loading characteristics as long as its uncertainty is considered in the study conclusions and recommendations.

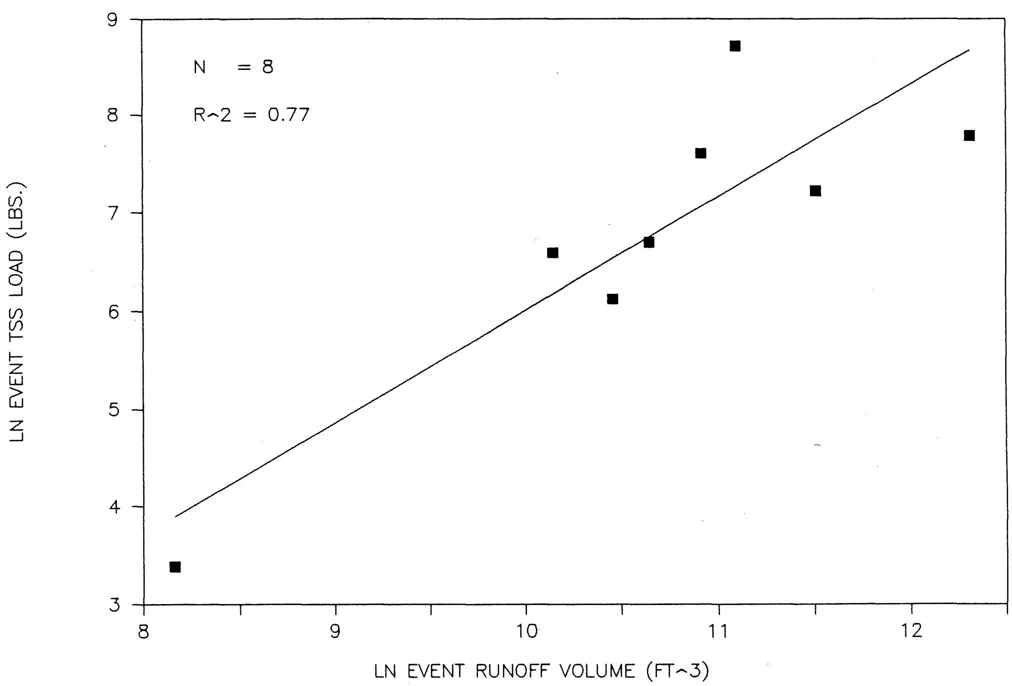


Figure IV-9. Simple Regression of Transformed Event Runoff Volume and Event Total Suspended Solids Load Data.

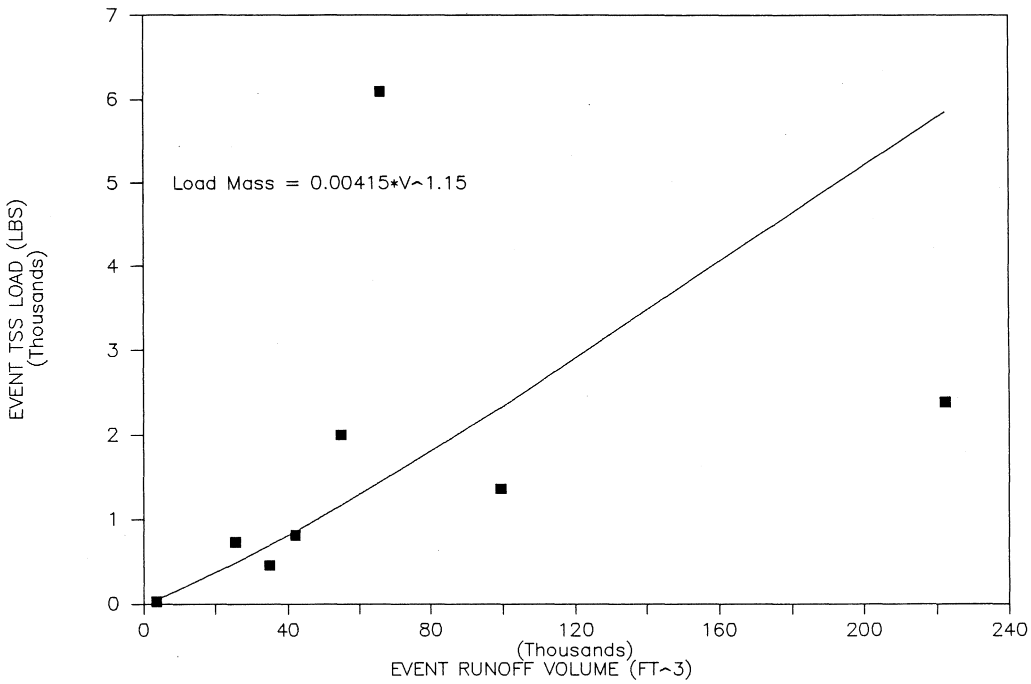


Figure IV-10. Relationship Between Event Runoff Volume and Event Total Suspended Solids Load.

CHAPTER V

HYDROLOGIC SIMULATION OF THE STUDY AREA

Introduction

Changes in land use in a watershed can significantly alter its natural hydrologic response. A quantified prediction of the change in hydrologic response for a specified change in land use is fundamental to engineering design and the assessment of environmental impact.

Relatively simple estimates of runoff as a function of land use are commonly performed for certain specified areas within the Indian River Lagoon catchment. Typically, these computations are performed for engineering design and are based on synthetic design storm events. Only two documented attempts at modeling large sub-catchments within the Indian River Lagoon basin were located. A preliminary assessment study of the 13 square mile Turkey Creek sub-catchment (Suphunvorranop and Clapp, 1984) used the STORM model (U.S. Army Corps of Engineers, 1977) to simulate total runoff hydrographs and pollutant concentrations. This model was selected because it was readily available and required less input data than other water quality models. Unfortunately because of a lack of continuous streamflow records and backwater effect from the lagoon, no calibrations were performed. In addition, the STORM model is limited in coastal areas of Florida as it does not include groundwater contributions to runoff. Da Costa and Glatzel (1987) conducted a simulation of "30 year monthly normal conditions" of runoff to the entire Indian River Lagoon using basic water budget methods outlined in Dunne and Leopold (1978). The major weakness of this approach is it lacks an accounting of groundwater discharge to the dense network of drainage canals, and such large time steps may be inadequate for receiving water analysis. Nevertheless, for Crane Creek, a small stream, they state that the agreement between field observations and simulation was within 10%; however, no data were presented.

Goodness of Fit Criteria

Deterministic hydrologic model accuracy is largely determined by (1) available input, (2) monitoring data on system response to varying inputs, and (3) the degree of model calibration and verification to multi-events over time. After considering problems associated with generating synthetic streamflow from models, Stedinger and Taylor (1982) concluded that the impact of parameter uncertainty is much greater than that of the selection between a simple and a relatively complicated model. They went on to recommend that given what is actually known about a basin's hydrology, model parameter uncertainty should be incorporated into simulation studies to obtain realistic and honest estimates of watershed response. However for many applications, it is highly impractical to obtain precise uncertainty information on what may be hundreds of parameters required to describe watershed response. Current practice assumes that the simplest model that will describe the system for the given input data should be used (Bedient and Huber, 1988). These are typically lumped parameter models that are calibrated to actual performance data.

Calibration involves minimization of deviations between measured field conditions and model output by adjusting parameters within the model. Verification is the process of checking the model calibration using an independent set of data.

Depending on the objectives, simulation of stormwater discharge, baseflow, and hydrologic/hydraulic control in a coastal drainage canal could require emphasis on the close prediction of total annual flow, average monthly flow volumes, peak flows, low flows, or specified events. However, Heaney et al. (1986) noted that equally successful prediction of each of these flow statistics using the same set of parameters is highly improbable. Sensitivity analysis for a lumped parameter model often shows a change in one parameter may improve one aspect of the calibration though reducing the "goodness of fit" of another aspect. Jewell et al. (1978) even found that different storm events resulted in different calibrations and different predictions. Therefore, to make decisions on changing parameter values during the calibration of the model, goodness of fit criteria need to be established based on the objectives of the study. The objectives for this simulation experiment were the minimization of the average differences between measured and predicted total event runoff volume, peak flows for specified events, and total event pollutant load.

This discussion has highlighted some of the limitations of watershed scale hydrologic simulation. Despite this, models still provide the most logical and scientifically advanced approach to understanding the hydrologic behavior of complex watershed and water resources systems (Bedient and Huber, 1988).

Runoff Block

The Storm Water Management Model (SWMM) simulates real storm events on the basis of rainfall and other meteorological inputs and system (catchment, conveyance, storage/treatment) characterization to predict outcomes in the form of quantity and quality values (Huber and Dickinson, 1988). The model consists of four main computational blocks (RUNOFF, TRANSPORT, Storage/Treatment, and EXTRAN) and six service blocks. The TRANSPORT, Storage/Treatment, and EXTRAN blocks may all use input and provide output to any block, including themselves.

The Runoff Block (RUNOFF) has been developed to simulate both quantity and quality runoff phenomena of a drainage basin and the routing of flows and contaminants to major conduits. Used in either the single event or continuous mode, the program takes a step-by-step accounting of rainfall, infiltration losses to pervious areas, surface detention, overland flow, soil moisture, evaporation, groundwater discharge to drainage canals, gutter flow, and the constituents washed into inlets, leading to the calculation of a number of inlet hydrographs and pollutographs (Huber and Dickinson, 1988). Sub-catchments 4 and 5 in the study area (see Figure IV-1, p. 34) were monitored, RUNOFF was calibrated and verified to these data and then applied to the remainder of the study area using Geographical Information System (GIS) data and other discretized data. These two sub-catchments are representative of developed areas of the study area. RUNOFF is most sensitive to parameters that describe the degree of development (e.g. impervious area, drainage channel geometry).

Runoff Quantity Data Input

Runoff Block parameters for sub-catchments 4 and 5 are listed in Table V-1. The average slope was found by

Table V-1. Runoff Block Parameters for Sub-Catchments 4 and 5.

PARAMETER	UNITS	CATCHMENT 4	CATCHMENT 5
Area	ac.	167	133
Characteristic width	ft.	4000	3000
Percent hydraulically connected imperviousness		18	15
Slope		0.00012	0.00012
Manning's roughness			
Impervious		0.013	0.013
Pervious		0.25	0.25
Channel		0.044	0.044
Horton parameters			
Hydraulic conductivity	in./hr.	5.0	5.0
Initial infiltration rate	in./hr.	10.0	10.0
Decay rate of infiltration	1/sec.	0.00115	0.00115
Conduit parameters			
Length	ft.	4000	4000
Width	ft.	15	15
Invert ope		0.00035	0.00035
Side slope		1/2	1/2
Full depth	ft.	4.0	4.0
Weir height	ft.	0.04	0.04
Groundwater parameters			
Porosity		0.3	0.3
Wilting point		0.03	0.03
Field capacity		0.05	0.05
HCO		10	10
PCO		15	15
Fraction of max. ET rate in upper zone		0.001	0.001
Deep recharge	in./hr.	0.00003	0.00003
Maximum depth ET	ft.	4.5	4.5

Note: HCO = Calibration parameter, estimates change in unsaturated hydraulic conductivity with change in soil moisture.

PCO = Change in soil tension/change in soil moisture.

calculating the path length between a point on the edge of the catchment to the catchment outlet, and dividing the path length by the change in elevation. The catchments essentially have a uniform, flat slope. Percent of hydraulically connected impervious area was based on a calibrated fraction of the total impervious area as determined by the updated Industrial Area GIS land use files. After calibration, the fraction was found to range from 0.33 to 0.38. For uncalibrated sub-catchments, 35% of the impervious area was assumed to be hydraulically connected.

Soils in the study area are primarily of the Spodosol Order with much of them having been classified as Urban type (Huckle et al. 1974) which originated mainly from Immokalee soils. Lesser areas of Felda, Basinger, Anclote, and Myakka soils are also found. Soil types and parameters were lumped and assumed to be represented by Immokalee soils. The Horton infiltration equation option was used. Maximum initial infiltration and the decay rate of infiltration were assumed to be represented by published data summarized in the SWMM manual (Huber and Dickinson, 1988) for dry sandy soils. The asymptotic infiltration rate is essentially equal to saturated hydraulic conductivity. For Immokalee soils, Carlisle et al. (1985) report a value of 4 in/hr. for saturated hydraulic conductivity. Using in-situ slug test procedures in the study area, Clark Engineers-Scientists, Inc. (1987) reported hydraulic conductivity ranging from 4 to 7.5 in/hr. A value of 5.0 in/hr. was selected for use in RUNOFF.

Manning's n values were selected from tables based on average type ground cover and from the rating curve development. Average monthly pan evaporation data were from National Weather Service values for Vero Beach, Florida published by Heaney et al. (1984). The catchment width parameter in RUNOFF represents the physical width of overland flow for the catchment; however, in lumped catchments it is used as a primary calibration factor. After calibration, a rule of thumb used to determine characteristic width for uncalibrated sub-catchments was to calculate it from catchment area with a catchment length of 1800 feet, assuming a rectangular catchment. Catchment length physically represents the distance overland flow must travel to reach a conduit (channel flow). A single conduit with a "dummy" mini-rectangular weir outfall was used for each sub-catchment to provide routing delay and account for in channel dead storage (due to depressions in the channel and culvert invert elevations being above channel invert elevations). The conduit length was used as a fine tuning calibration parameter; however, it was based on the length of the trunk or main drainage channel of the catchment.

In Florida, where land slopes are flat and water tables are high, the primary drainage pathway is often through the surficial aquifer and the unsaturated zone above it. Capece et al. (1984) reported that in Florida Flatwood areas a storm will cause a rise in the water table which results in a natural detention release to the drainage canals. A simple estimate by Clark Engineers-Scientists, Inc. (1987) indicated that groundwater discharge to drainage canals accounted for 11% of the water budget for KSC while direct runoff accounted for less than 2%. This process was also observed during the data collection phase of this project and later hydrologic simulation experiments suggested that this phenomenon was responsible for a large percentage of the runoff from the study area (see simulation results section).

RUNOFF, in the latest version of SWMM (Version 4), contains a subroutine to account for subsurface flow routing (groundwater seepage to canals). The groundwater subroutine simulates two zones, an upper unsaturated zone and a lower saturated zone. Inflow into the groundwater subroutine is calculated in the infiltration subroutine. The flow from the unsaturated to the saturated zone is controlled by a percolation equation. Losses and outflow from the lower zone can be via deep groundwater recharge, saturated zone evapotranspiration, and groundwater flow. Groundwater flow is a user-defined power function of water table stage and, if chosen, depth of water in the discharge channel (Huber and Dickinson, 1988).

The general groundwater discharge equation provided by the model is as follows:

$$GWFLW = A1 * (D1-BC)^{B1} - TWFLW + A3 * D1 * TW \quad (7)$$

and

$$TWFLW = A2 * (TW-BC)^{B2} \quad (8)$$

where

GWFLW = beginning of time step groundwater flow rate (per area),

TWFLW = channel water influence flow rate (per area),

A1,A2 = groundwater and channel water influence coefficients,

A3 = coefficient for cross-product,

B1,B2 = groundwater and tailwater influence exponents,

BC = elevation of bottom of channel,

TW = elevation of water in channel, and

D1 = average water table head.

This general equation was given the functional form of the Dupuit-Forcheimer approximation (Bouwer, 1978) for drainage to an adjacent channel by setting flow coefficients and flow exponents as follows:

$$A1 = A3 = 4K/L^2 \quad (9)$$

and

$$A2 = 0, \; B1 = 2$$

where K = hydraulic conductivity = 10 ft./day (5in./hr.)

L = length of flow for groundwater influenced by channel.

Based on observations at KSC reported by Clark Engineers-Scientists, Inc. (1985), L ranges from 300 to 500 feet. Therefore, the reasonable range for A1 and A3 is from 2.22E-4 to 8.0E-5 (in./hr.-ft.^2). During the calibration process, a value of 1.2E-4 (in./hr.-ft.^2) was found to provide acceptable results.

Initial upper zone moisture content and water table elevation for calibration events were estimated using continuous simulation prior to the event. Initial estimates of porosity, field capacity, and wilting point 0.46, 0.13,and 0.038, respectively) were made from published data (Carlisle et al., 1985) for Immokalee soils. However, such data produced poor matches of channel baseflow, the observed hydrograph characteristics indicative of groundwater flow (slow prolonged discharge at the tail of the stormwater runoff hydrograph), and groundwater table fluctuations. Therefore these parameters in effect also became calibration parameters. As a guide in estimation, Bennett's (1988) lumped parameter estimates were used. Using a calibrated, lumped parameter spreadsheet simulator to predict water table behavior in Merritt Island, Florida, Bennett (1988) found values for porosity and field capacity of 0.2 and 0.0275, respectively, to provided good matches between observed and simulated groundwater table fluctuations. Percolation calibration parameters used in the SWMM manual for an example for the Cypress Creek, Florida watershed were also used for this study area. The KSC Groundwater Survey

(Clark Engineers-Scientists, Inc., 1987) provided data on (1) the rate of evapotranspiration (ET) from the water table, (2) the depth limit of ET from the soil, and (3) the rate of "effective" surficial aquifer recharge.

Calibration and Verification.

A randomly selected mix of event periods, generally distributed over the 1988 year, was used for calibration and verification. Calibration was done for average conditions across several storms by maintaining the same parameter values for each until predictive error was a minimum for all events. From sensitivity analyses, it was deduced that percentage impervious area, and groundwater parameters would be the primary volume calibration factors.

Characteristic width and routing conduit parameters change the shapes of the hydrographs and improve and predicted peak flows. The agreement between measured and predicted volumes and peak flows for Catchment 4 are shown in Figures V-1 and V-2. The agreement for Catchment 5 are shown in Figures V-3 and V-4. Some Catchment 4 multi-event hydrographs of measured and predicted flows from the calibration and verification runs are shown in Figures V-5 and V-6; similar hydrographs for Catchment 5 are shown in Figures V-7 and V-8. Comparisons of measured and predicted groundwater hydrographs for Catchments 4 and 24 are shown in Figures V-9 and V-10, respectively.

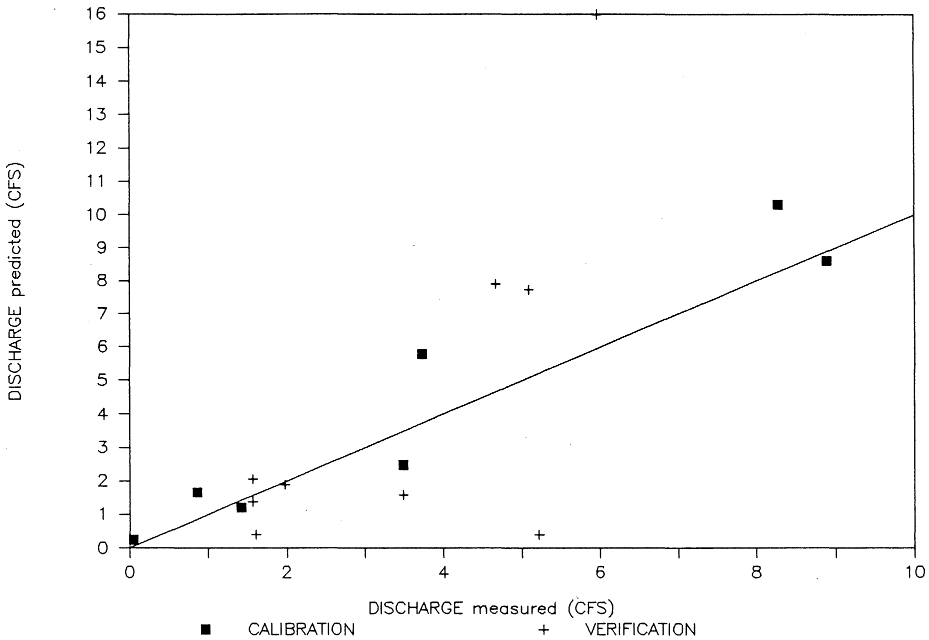


Figure V-1. Peak Flow Results from the Calibration and Verification Runs--Catchment 4.

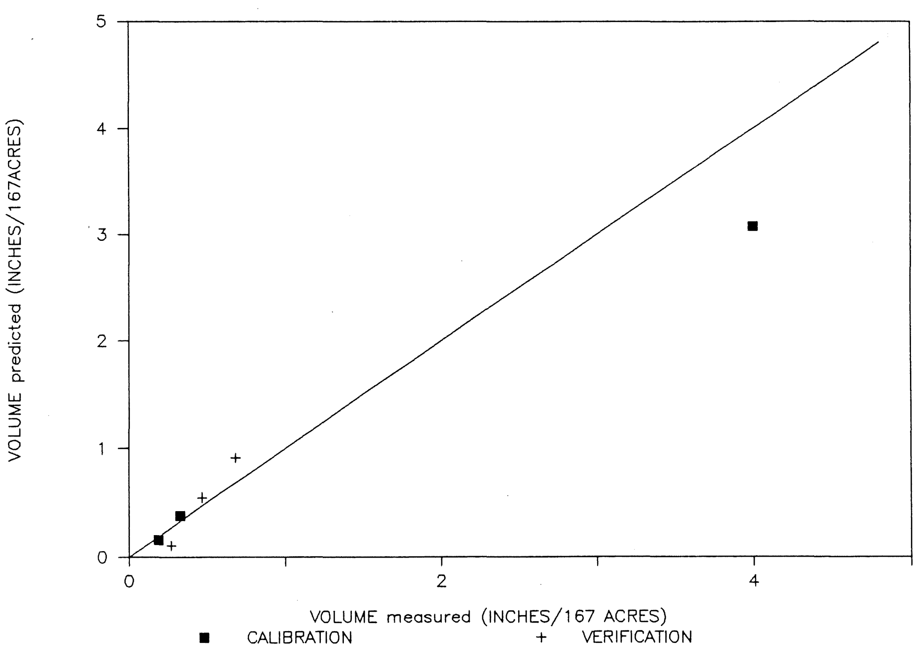


Figure V-2. Total Flow Results From the Calibration and Verification Runs--Catchment 4.

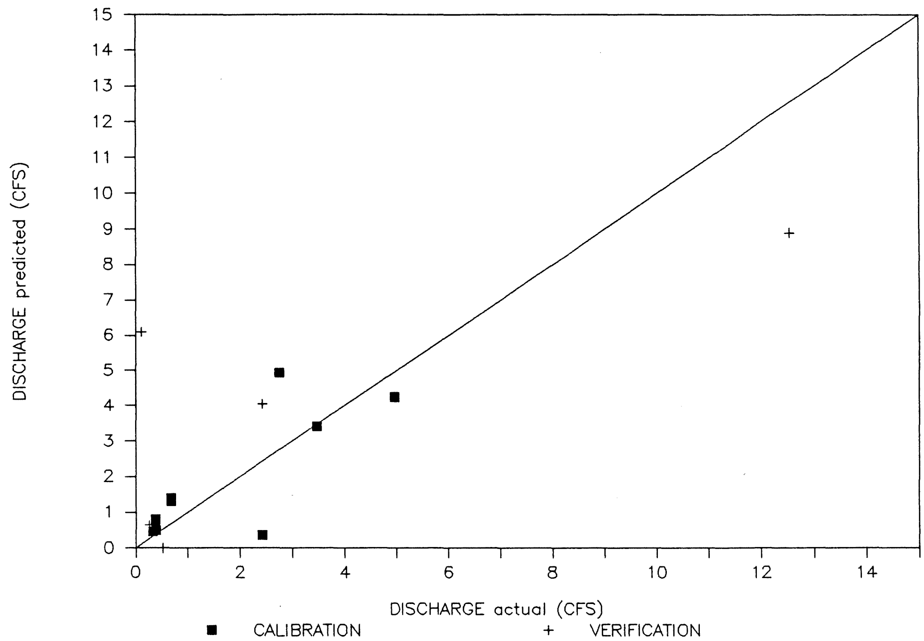


Figure V-3. Peak Flow Results from the Calibration and Verification Runs--Catchment 5.

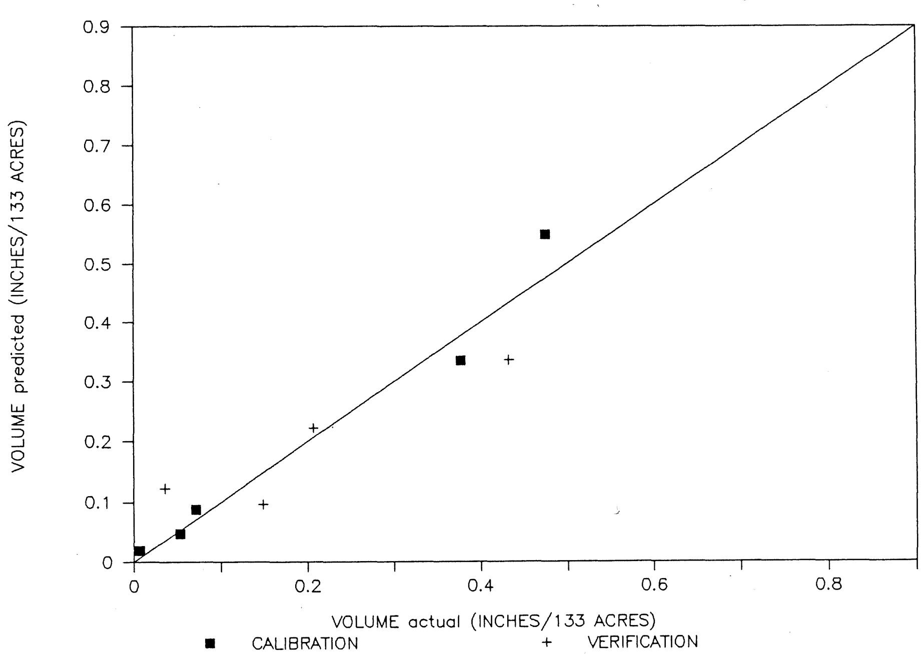


Figure V-4. Total Flow Results from the Calibration and and Verification Runs--Catchment 5.

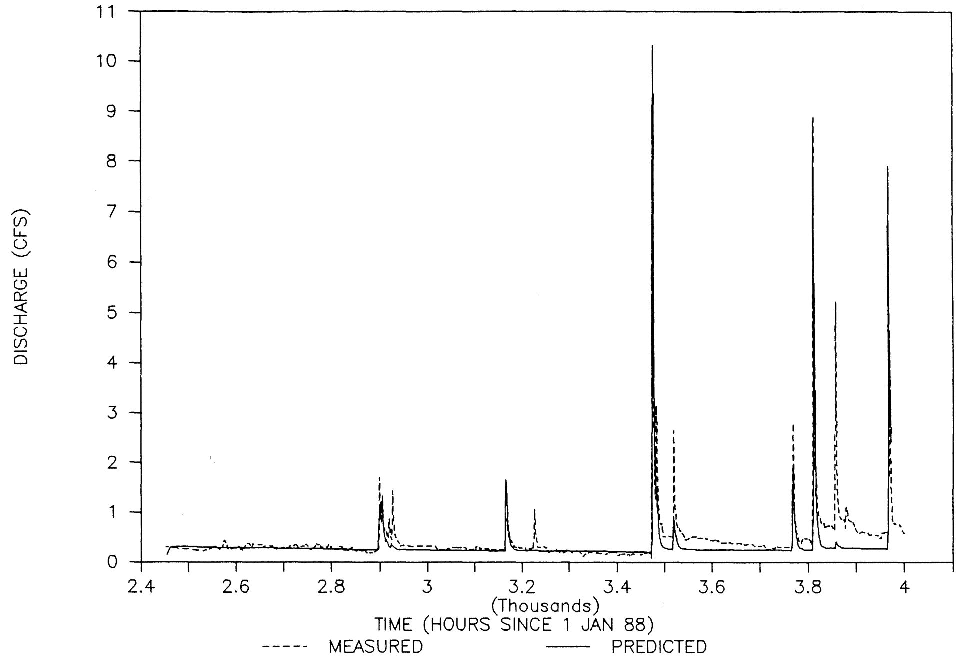


Figure V-5. Measured and Predicted Hydrographs from a Calibration Run on Catchment 4. April 12, 1988 through June 15, 1988.

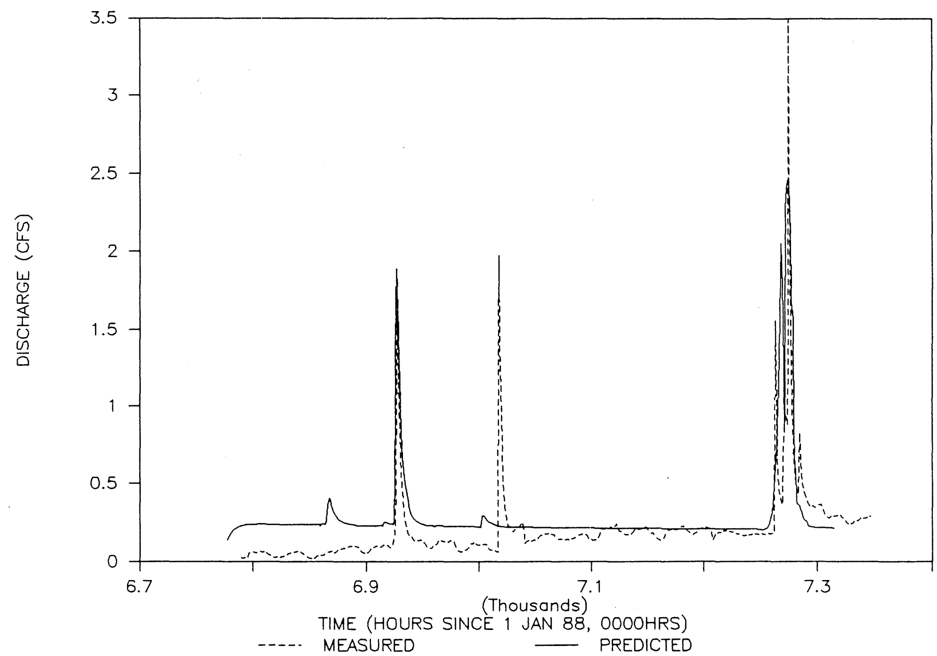


Figure V-6. Measured and Predicted Hydrographs from a Verification Run on Catchment 4. October 10, 1988 through November 2, 1988.

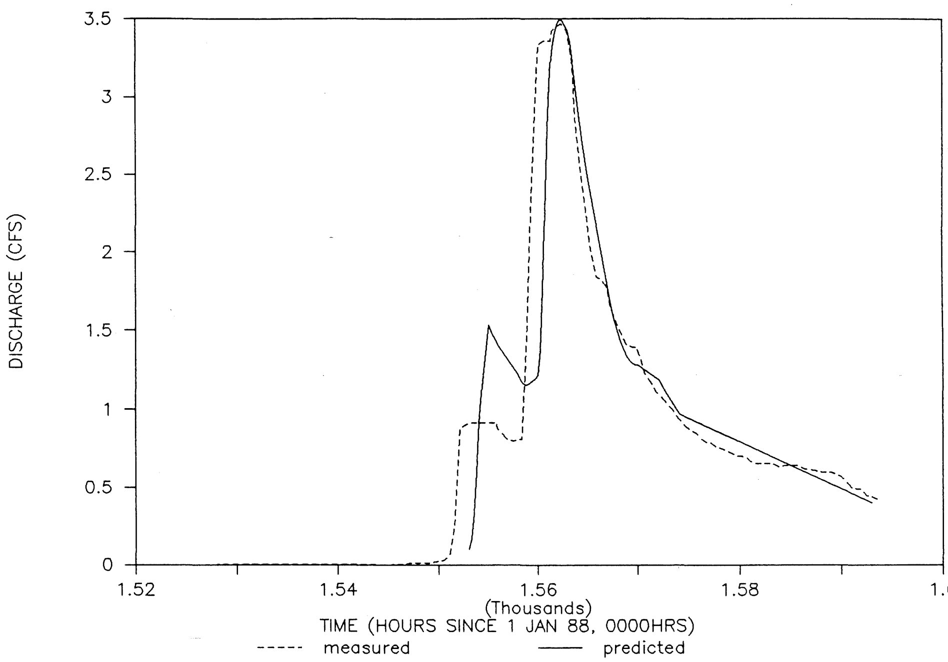


Figure V-7. Measured and Predicted Hydrographs from a Calibration Run on Catchment 5. March 4, 1988 through March 7, 1988.

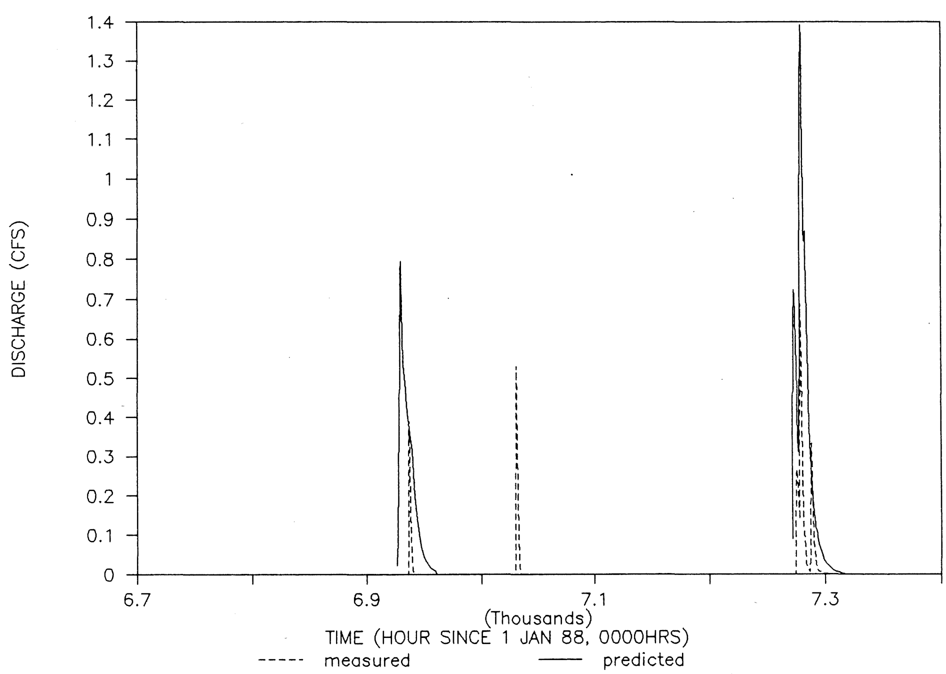


Figure V-8. Measured and Predicted Hydrographs from a Verification Run on Catchment 5. October 10, 1988 through November 2, 1988.

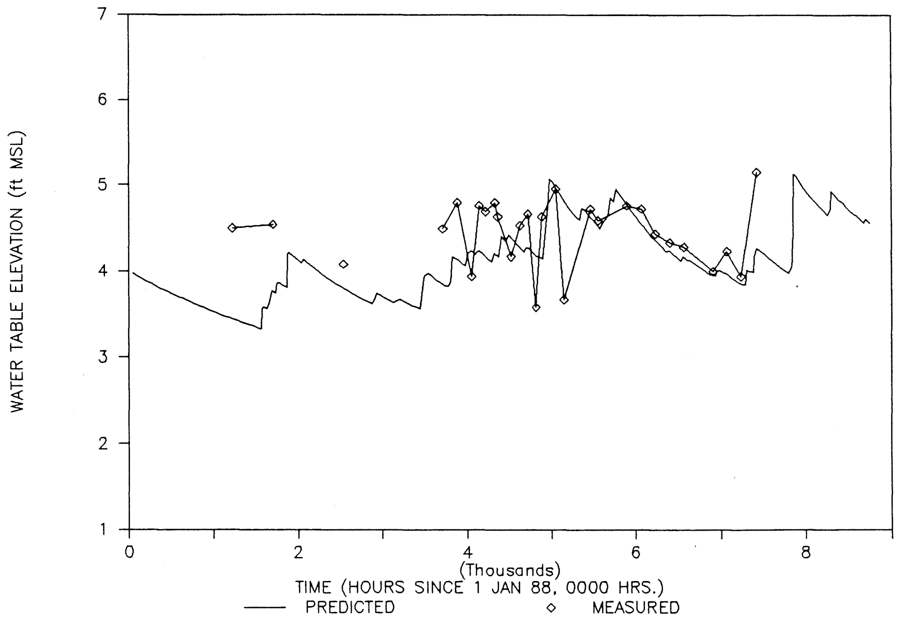


Figure V-9. Predicted and Measured Groundwater Table Elevation for Observation Well BAS18 Located in Catchment 4.

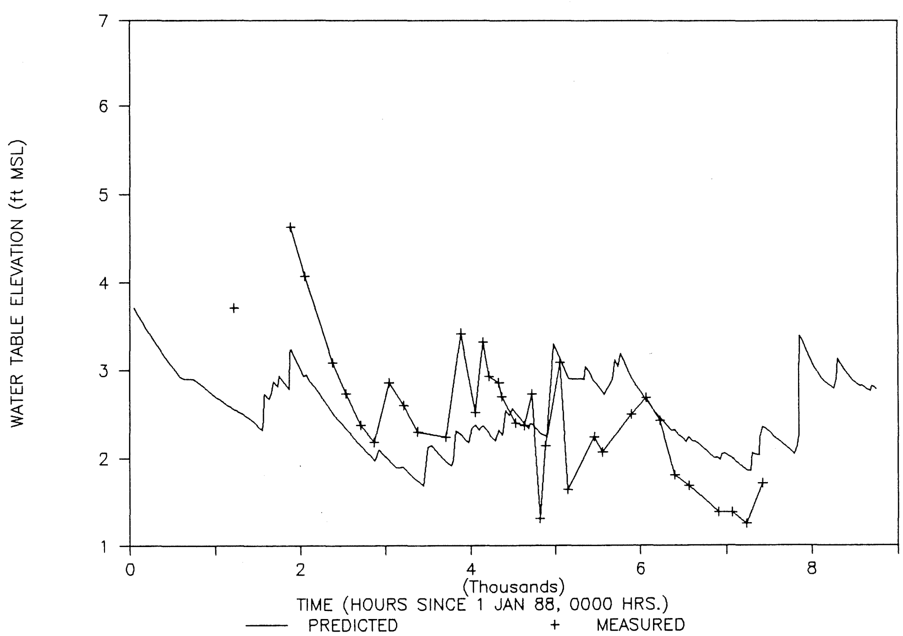


Figure V-10. Predicted and Measured Groundwater Table Elevation for Observation Well BAS20 Located in Catchment 24.

It is hypothesized that a major portion of the error is accountable by variable rain distribution over the catchment, error in using rain data 1.1 miles from the study area, and exfiltration of runoff from channels during the dry season. Other error may be introduced by the model's simplistic representation of groundwater discharge to the channels. Noted limitations are the lack of accounting of exfiltration from the channels.

Runoff Quality

Available deterministic models usually compute stormwater quality via a series of steps. First accumulation of pollutants during dry weather buildup is estimated. Then estimates of the accumulated pollutants that are washed off (washoff) is made. Since the processes of accumulation, redistribution, washoff, transport, and dispersal are complex in nature, a general model for the complete process suitable for application to all catchments has yet to be developed (James and Boregowda, 1985). However, with sufficient site-specific calibration data, reasonable predictions can be made using empirical relationships (e.g. rating curves) or simple, physically-based functions found in most general water quality models (Huber, 1985).

Data Input

Water quality parameters used in RUNOFF are listed in Table V-2. Three buildup equation forms are available in RUNOFF, power-linear, exponential, and Michaelis-Menton. A linear, no upper limit buildup formulation for particulate was selected as a starting point for pollutant buildup calibration for this study. Because development in the study area is relatively mature and observation suggested that channel scour was the principal contributor of suspended solids, the functional dependence of the buildup parameters was based on channel length rather than catchment area. For pollutants other than particulates, this approach would not be appropriate (e.g. nutrients, metal, and hydrocarbons are better related as a function of catchment area). Channel length for pollutant buildup was based on total channel length rather than main channel length. To account for ambient concentrations of TSS, it was assumed that groundwater flow contributed 1 mg/L of TSS. The number of dry days before a rain event was estimated to be 4.0 days; this corresponds to the mean interevent rain period for Melbourne, Florida (Heaney et al. 1984). The buildup parameters were used for calibration of RUNOFF water quality predictions. Washoff estimates of suspended solids were based on the rating curve developed from study area data (see Chapter IV).

Table V-2. Runoff Block Water Quality Parameters.

PARAMETER	VALUE
Buildup	
No. of dry days before the start of storm	4
Type of buildup calculation (KCALC)	Power-linear (1)
Functional dependence of buildup parameters (KACGUT)	Gutter length (0)
Buildup limit	1.0E+6 lbs/100 ft.
Buildup exponent	1.0
Buildup coefficient	5.0
Washoff	
Type of washoff calculation (KWASH)	Power (0)
Exponent for washoff rate (WASHPO)	1.15
Washoff rate coefficient (RCOEF)	40

Note: Parentheses indicate SWMM input variable nomenclature.

Calibration

For water quality predictions, calibration of RUNOFF centered around a trial-and-error procedure to determine the ideal combination of buildup rate coefficients that result in a satisfactory match of the measured and predicted event suspended solids loads. RUNOFF was calibrated for average conditions across several storms to reduce predictive error and increase confidence. The agreement between measured and predicted event loads of suspended solids is shown in Figure V-11. The loading predictions look reasonable with the exception of event TSS5-4 which appears to be an outlier. Channel scraping (cleaning) during the week of this event explains the extraordinarily high loading measured. The presence of outliers in a data set creates analysis problems. They distort statistical measures and, in this case, the outliers are generally neither periodic, nor predictable. Fortunately, this outlier is explainable and, therefore excluded from the calibration analysis as it is unrepresentative of "average conditions".

Variations in the runoff quantity predictions contribute to error in the predicted loads. For example, a slight difference between observed and predicted runoff discharge concurrent with a high rate of mass transport, contributes further to the error in estimating pollutant loads. Because the goal of this project was to estimate the loadings to the estuary and to examine relative

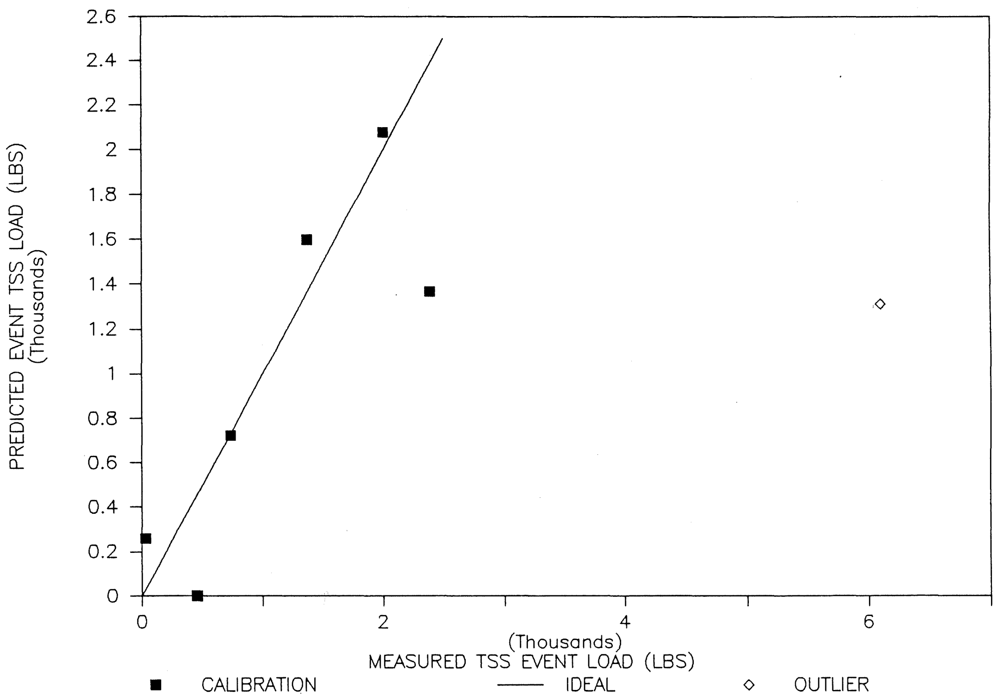


Figure V-11. Goodness of Fit of Event Total Suspended Solids Load.

effectiveness of multi-objective surface water management strategies, the model results were deemed acceptable.

Estimates of Loadings from the Industrial Area Catchment

An objective of this study was to estimate typical annual freshwater discharge (from both overland runoff and groundwater seepage to channels) and pollutant loadings (represented by TSS) under existing land-use conditions and future development for the study area. Future research plans include incorporating such estimates in a receiving water response analysis.

From the environmental management perspective, receiving water may respond slowly to input (particularly estuaries where the response time has been estimated at days to weeks (Driscoll, 1979)). It is seldom necessary to consider more than the total pollutant load from a storm for receiving water analysis. Detailed pollutographs are usually not required since short time-step variations are strongly dampened by the receiving water response (Huber, 1985). For example, Bartleson (1988) suggests that high concentrations of dissolved nutrients can increase epiphytic algae coverage and reduce light, thus decreasing seagrass growth. However, short term pulses of nutrients or turbidity may have little effect on the seagrass biomass. Stresses applied relatively evenly over large estuarine areas will be the most difficult for seagrass to tolerate (Florida Department of Environmental Regulation, 1987).

Data Input

Calibrated input files for sub-catchments 4 and 5 were extrapolated to represent the entire study area using rules developed during calibration. A schematic diagram catchment and associated channel numbers as represented in the model of the study area is shown in Figure V-12. Parameters varying between catchments under existing conditions are summarized in Table V-3. Since "typical" loadings to the receiving water are sought, a continuous simulation of existing development was driven by the hourly precipitation record for 1951 at the Melbourne station. As noted in Chapter IV, 1951 was a "typical" rainfall record year. Sub-catchments 3 and 24 were simulated as "shoreline" catchments where the average surface elevation is approximated at 5.0 ft MSL and the initial water table elevation is 3.7 ft. MSL. The channel for sub-catchment 1 is heavily vegetated (not maintained) and is assumed to produce little TSS loading due to scour. Therefore, channel 111 for sub-catchment 1 is assumed to have only 100 feet of its 6200 feet available for "pollutant buildup".

The timing of discharges to the receiving water is controlled by the backwater conditions of the lagoon. Because of a lack of long-term lagoon elevation data, "typical" lagoon water level elevation (backwater elevation) as a function of time could not be generated. Therefore continuous hydraulic routing of runoff to the lagoon was not

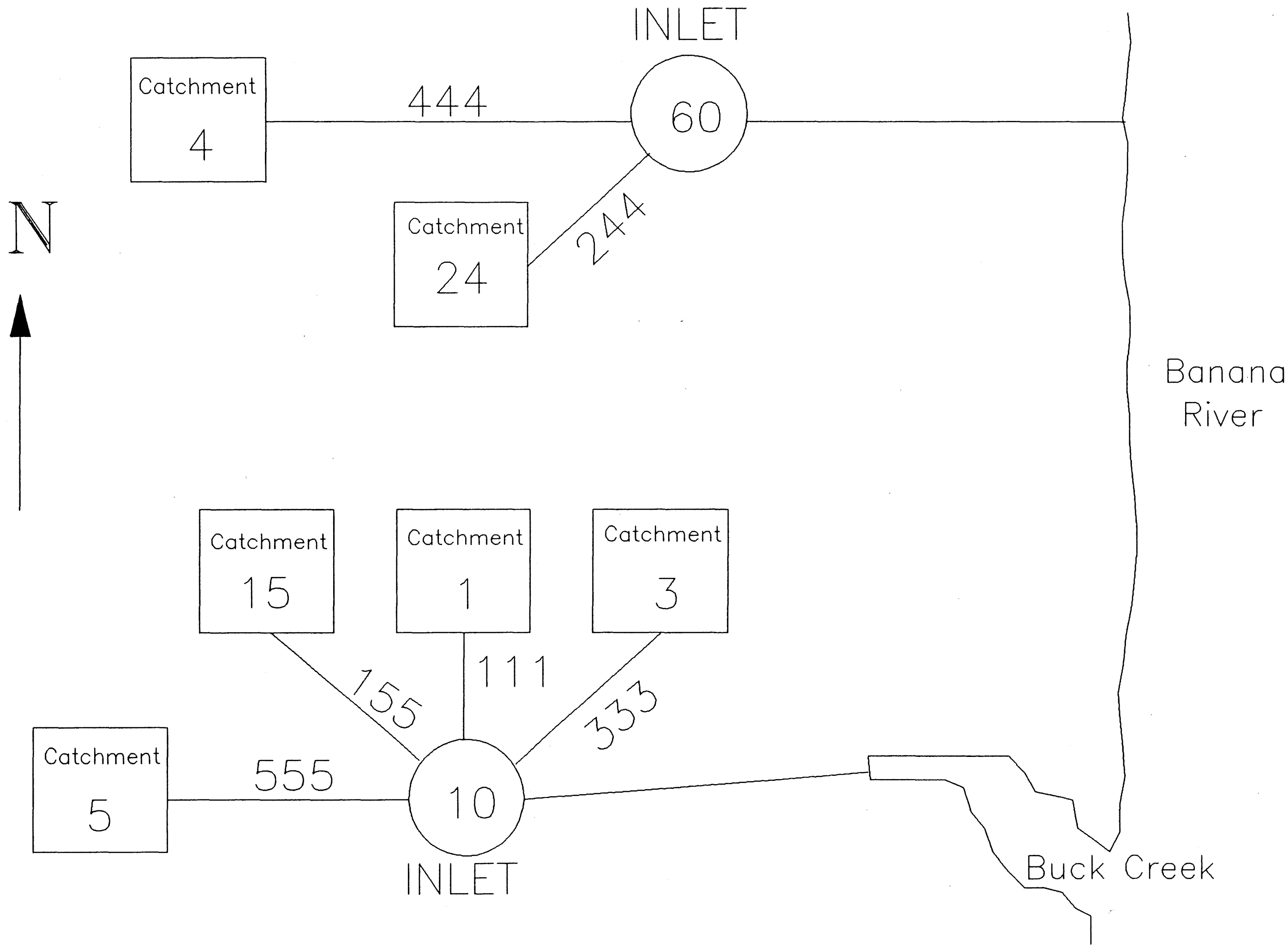


Figure V-12. Schematic of the RUNOFF Simulation for the Entire Study Area.

Table V-3. Runoff Block Parameters for the Study Area.

Sub-Catchment	Area (ac.)	Characteristic Width (ft.)	Hydraulically Connected Imperviousness (%)	Channel Length (ft.)	Initial Water Table Elev. (ft. MSL)	Avg. Grade Elev. (ft. MSL)
1	577.2	13,970	0.0	6,200	4.5	8.0
3	489.3	11,841	4.5	4,000	3.7	5.0
4	167.0	4,000	18.0	4,000	4.5	8.0
5	133.0	3,000	15.0	4,000	4.5	8.0
15	218.5	5,290	15.0	3,600	4.5	8.0
24	315.6	7,640	5.0	6,000	3.7	5.0

attempted. Residence time in the trunk channels is estimated at less than 2 hours under the existing configuration. However, no adjustment factor was applied to account for deposition in the trunk channel areas influenced by backwater. Although a relatively crude technique, it represents the best available procedure to roughly estimate solids loadings to the estuary until long-term data sets on lagoon water levels are generated. Detailed hydraulic analysis of TSS transport through the channels is futile until backwater conditions become predictable.

The processes involved in the transport of suspended solids at channel mouths to the lagoon are complex. Transport is influenced by such factors as the riverine-like discharges from the channels, quasi-periodic aeolian tides, and saline water enhanced coagulation of hydrophobic colloids (and sedimentation) in the sediment laden freshwater flows into the estuary (the double-layer compression aggregation mechanism (Manahan, 1984)). Exports will change as the hydrology is switched from a stream dominated system during wet-weather to an aeolian tide dominated system. However, after reviewing the Ryan and Goetzfried (1988) dry-weather data at the mouth of Buck Creek, suspended solids transport is dominated by wet-weather events. Unfortunately, this area of the drainage network can only be addressed qualitatively in this study until additional data are generated.

Future land use change in the study area and the time frame for any change are highly unpredictable. Therefore for these simulation experiments, the concept of maximum buildout is presented. The definition of maximum buildout is somewhat subjective and will likely vary between agencies and engineering/planning offices. For the purposes of this research, maximum buildout is based on the existing development in sub-catchments 4 and 5. It is assumed that these catchments are fully developed. Calibration of the model suggests that 18% of a "fully developed" catchment will be hydraulically connected impervious area. The construction of additional drainage channels was estimated based on existing facilities layouts. The projection of buildup channel length was based on the drainage channel density in sub-catchments 4, 5, and 15. Details of this projection are found in Table V-4. A summary of existing and "maximum buildout" conditions in the study area is shown in Table V-5. The actual RUNOFF input files used are found in Appendix B.

Groundwater discharge to the drainage network can be expected to increase with increased development (maximum buildout). However in RUNOFF, groundwater discharge is calculated as a function of catchment area. This approach does not represent the length of the actual seepage face. Therefore the increase in seepage face length expected with increased development was not physically represented. Any difference in the predicted groundwater discharge between

Table V-4. Calculations of Channel Buildup Length for Water Quality Simulation in RUNOFF.

Assumptions:

Catchments 4, 5, and 15 represent typical buildout density and drainage channel density.

Catchment	Area (ac.)	Existing Buildup Channel Length (ft.)	Drainage Density (ft./acre)
Catchment 4	167	13,800	82.6
Catchment 5	133	8,700	65.4
Catchment 15	218.5	16,200	74.1
		Mean Drainage Density =	74

Therefore, the formula for determining maximum buildout buildup channel length is as follows:

((Catchment Area) - Unbuildable Catchment Area) * 74 ft./acre

Unbuildable Catchment Area = Wetlands and Surface Water.

Catchment	Unbuildable Area (ac.)	Buildable Area (ac.)	Maximum Buildout Buildup Channel Length (ft.)
1	10	567	41,958
3	120	369	27,306
4	0	167	13,800
5	0	133	8,700
15	0	218.5	16,200
24	30.5	285	21,090

Table V-5. Development Scenarios for the Industrial Area.

PARAMETER	EXISTING	MAXIMUM BUILDOUT
SUB-CATCHMENT 1:		
Area	577.2 ac.	577.2 ac.
Percent hydraulically connected imperviousness	0.0	18
Channel length	6200 ft.	9000 ft.
Buildup channel length	100 ft.	42,958 ft.
SUB-CATCHMENT 3:		
Area	489.3 ac.	489.3 ac.
Percent hydraulically connected imperviousness	4.5	18
Channel length	4000 ft.	5000 ft.
Buildup channel length	6600 ft.	27,300 ft.
SUB-CATCHMENT 4:		
Area	167 ac.	167 ac.
Percent hydraulically connected imperviousness	18	18
Channel length	4000 ft.	4000 ft.
Buildup channel length	13,800 ft.	13,800 ft.
SUB-CATCHMENT 5:		
Area	133 ac.	133 ac.
Percent hydraulically connected imperviousness	15	18
Channel length	4000 ft.	4000 ft.
Buildup channel length	8700 ft.	8700 ft.
SUB-CATCHMENT 15:		
Area	218.5 ac.	218.5 ac.
Percent hydraulically connected imperviousness	15	18
Channel length	5600 ft.	5600 ft.
Buildup channel length	10,200 ft.	10,200 ft.
SUB-CATCHMENT 24:		
Area	315.6 ac.	315.6 ac.
Percent hydraulically connected imperviousness	5	18
Channel length	6000 ft.	7640 ft.
Buildup channel length	4000 ft.	21,000 ft.

existing and maximum buildout conditions has no physical meaning. In hindsight, groundwater discharge can be calculated as a function of channel length rather than catchment area by incorporating seepage face length in the discharge coefficients in the model's general groundwater discharge equation. This coefficient development is shown in Appendix C.

Results

The predicted annual water budget for the study area drainage network under existing land use conditions is shown in Figure V-13. Groundwater inflow accounts for 87% of the annual discharge from the study area. However, its contribution varies both within the catchment and within the storm event. For example, Figures V-14 and V-15 show groundwater discharge in upland and lowland subcatchments. During wet-weather, groundwater discharge is reduced if not discontinued. Combined discharge from the study area for a typical year is shown in Figure V-16. A comparison of predicted annual loads for existing land use and maximum buildout is shown in Table V-6 and by month in Figures V-17 and V-18. The model estimates that if development of the study area were to occur to its maximum potential, the annual discharge of freshwater would increase by 18% and the annual TSS load would increase by 290%.

```
***********************************************
*       CONTINUITY CHECK FOR CHANNEL/PIPES    *
***********************************************

                                                                INCHES OVER
                                                 CUBIC FEET     TOTAL BASIN
INITIAL CHANNEL/PIPE STORAGE................    2.900720E+05      0.042
FINAL CHANNEL/PIPE STORAGE..................    7.393456E+04      0.011
SURFACE RUNOFF FROM WATERSHEDS..............    1.544678E+07      2.239
GROUNDWATER SUBSURFACE INFLOW...............    9.651181E+07     13.990
EVAPORATION LOSS FROM CHANNELS..............    2.246527E+06      0.326
CHANNEL/PIPE/INLET OUTFLOW..................    1.107775E+08     16.057
INITIAL STORAGE + INFLOW....................    1.122487E+08     16.271
FINAL STORAGE + OUTFLOW.....................    1.130979E+08     16.394
********************************************
* FINAL STORAGE + OUTFLOW  + EVAPORATION - *
* WATERSHED RUNOFF - GROUNDWATER INFLOW  - *
*     INITIAL CHANNEL/PIPE STORAGE         *
*     ----------------------------         *
*  FINAL STORAGE + OUTFLOW  + EVAPORATION  *
********************************************
ERROR.......................................          0.751 PERCENT


***************************************************
*     CONTINUITY CHECK FOR SUBSURFACE WATER       *
***************************************************
                                                                INCHES OVER
                                                 CUBIC FEET     SUBSURFACE
                                                                BASIN

TOTAL INFILTRATION                              2.876568E+08    41.697
TOTAL UPPER ZONE ET                             4.080912E+05     0.059
TOTAL LOWER ZONE ET                             2.030482E+08    29.432
TOTAL GROUNDWATER FLOW                          9.651181E+07    13.990
TOTAL DEEP PERCOLATION                          1.146781E+06     0.166
INITIAL SUBSURFACE STORAGE                      1.117154E+08    16.193
FINAL SUBSURFACE STORAGE                        1.149681E+08    16.665
UPPER ZONE ET OVER PERVIOUS AREA                4.080912E+05     0.063
LOWER ZONE ET OVER PERVIOUS AREA                2.030482E+08    31.426


******************************************
* INFILTRATION + INITIAL STORAGE - FINAL *
* STORAGE - UPPER AND LOWER ZONE ET -    *
* GROUNDWATER FLOW - DEEP PERCOLATION    *
* -------------------------------------- *
*     INFILTRATION + INITIAL STORAGE     *
******************************************
ERROR ..................................          -4.184 PERCENT
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Figure V-13. Predicted Annual Water Budget for the Study Area Under Existing Land Use Conditions.

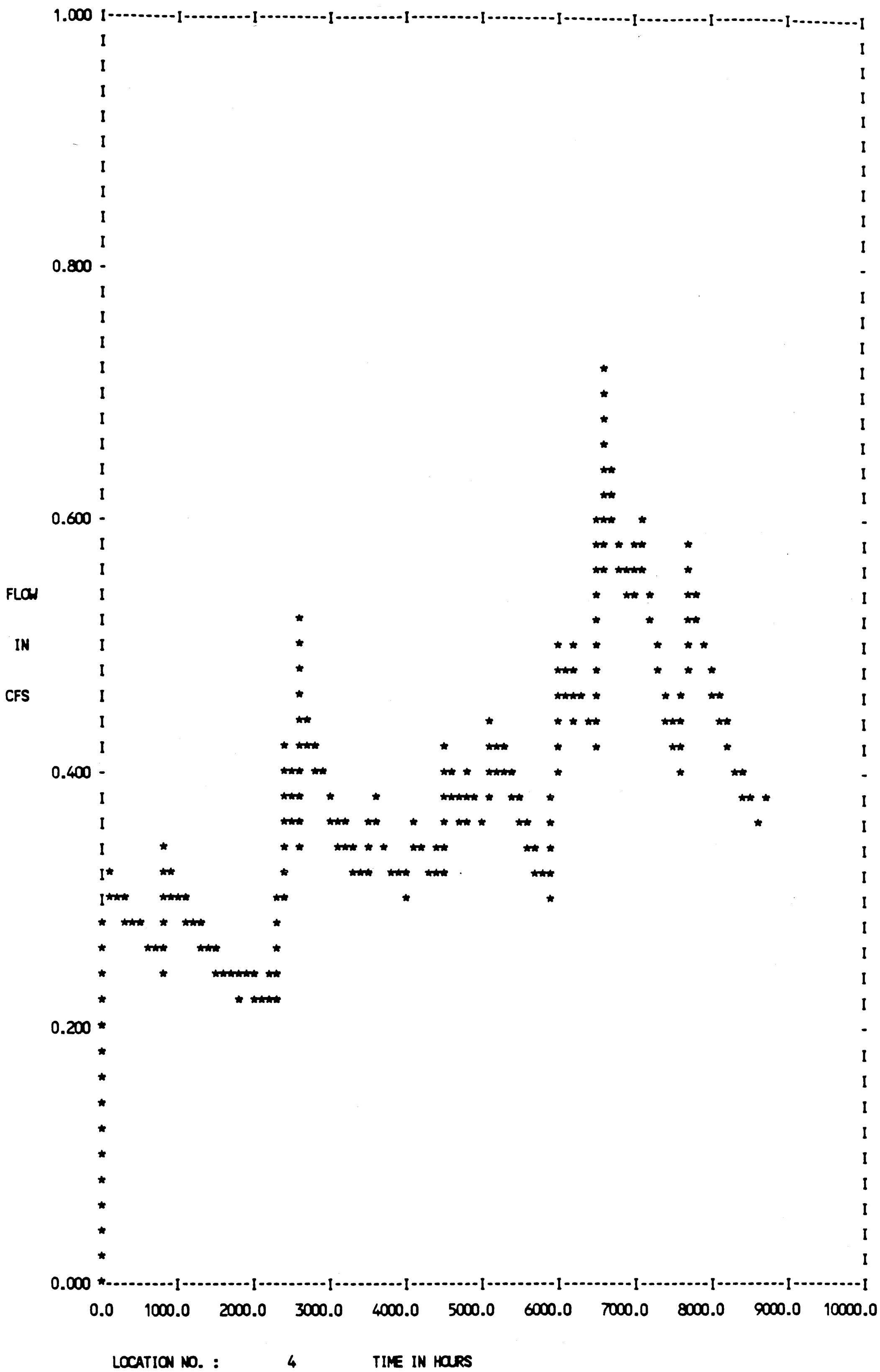

Figure V-14. Groundwater Discharge under Existing Land Use in One Upland Subcatchment for a "Typical" Year.

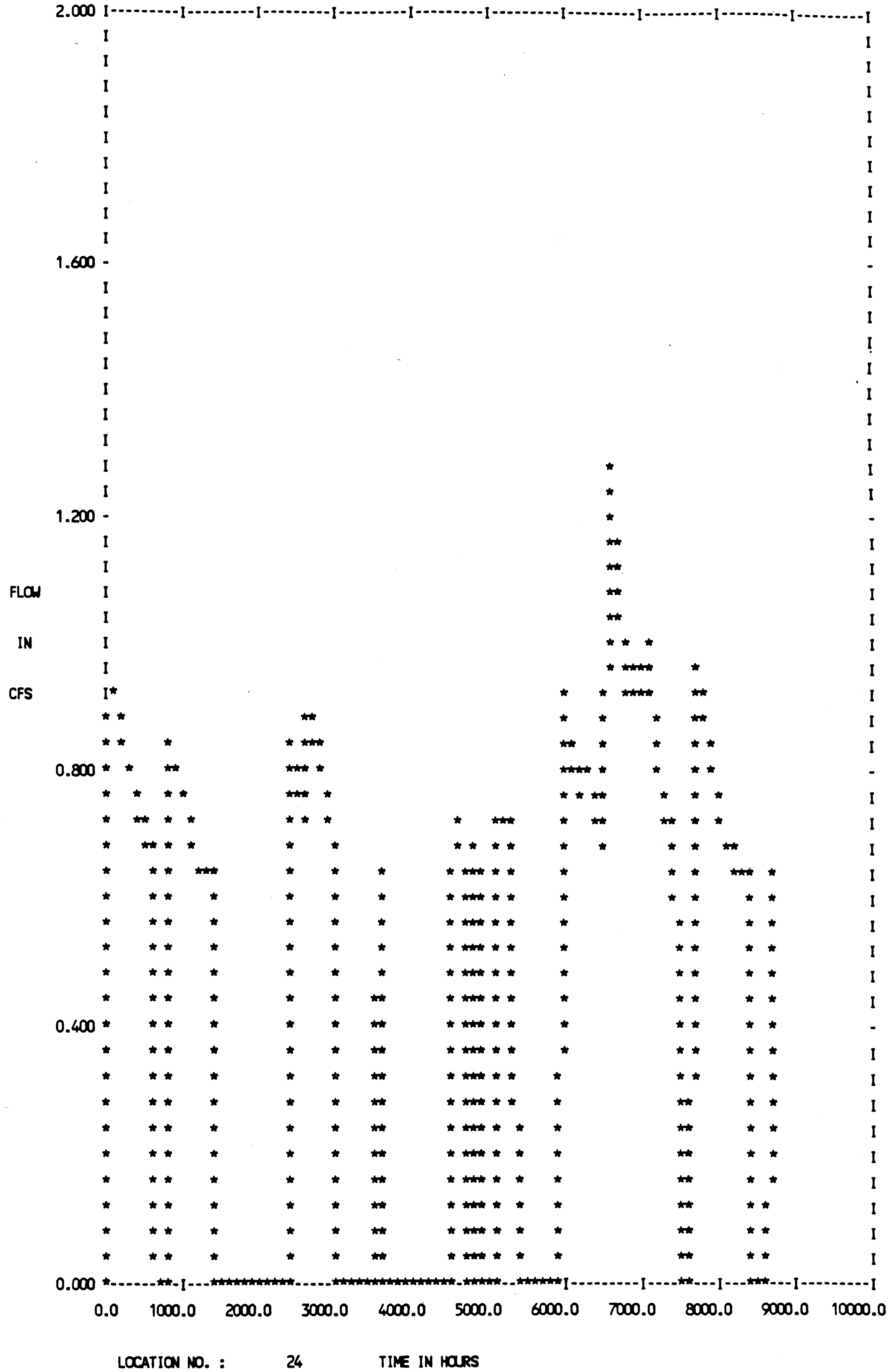

Figure V-15. Groundwater Discharge Under Existing Land Use in One Lowland Subcatchment for a "Typical" Year.

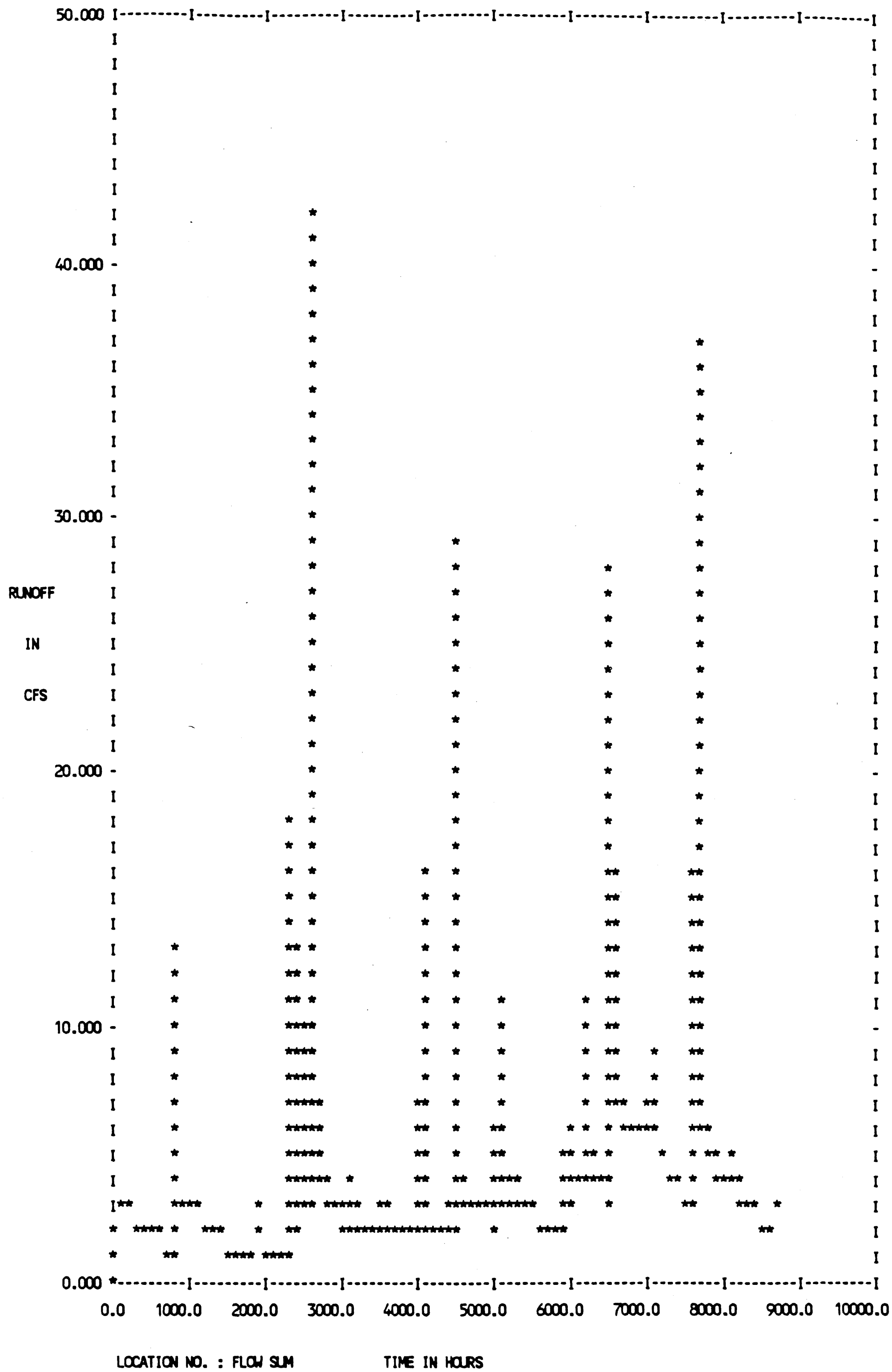

Figure V-16. Combined "Typical" Year Discharge from the Study Area Under Existing Land Use.

Table V-6. Predicted Annual Loads to the Indian River Lagoon From the Study Area.

ANNUAL LOADS		EXISTING LAND USE	MAXIMUM BUILDOUT
Combined freshwater discharge			
	Inlet 10	1819 ac-ft	2195 ac-ft
	Inlet 60	724 ac-ft	821 ac-ft
	Total	2543 ac-ft	3016 ac-ft
Groundwater discharge			
	Inlet 10	1628 ac-ft	1519 ac-ft
	Inlet 60	588 ac-ft	555 ac-ft
	Total	2216 ac-ft	2074 ac-ft
Total suspended solids			
	Inlet 10	357,600 lbs	1,304,000 lbs
	Inlet 60	272,400 lbs	581,000 lbs
	Total	630,000 lbs	1,885,000 lbs

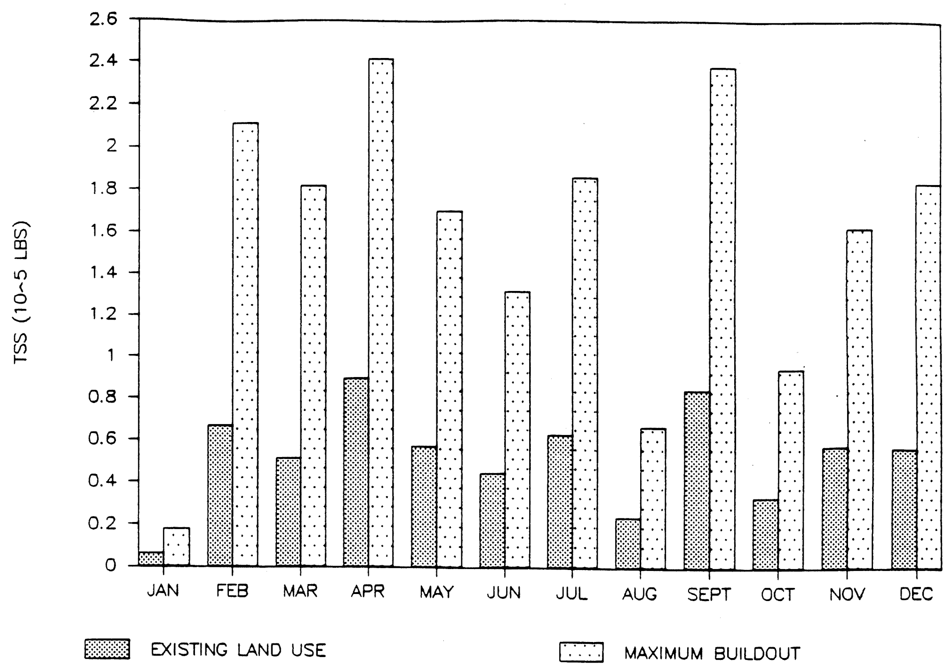

Figure V-17. "Typical" TSS Loads From the Study Area Under Existing Land use and the Maximum Buildout Development Scenarios.

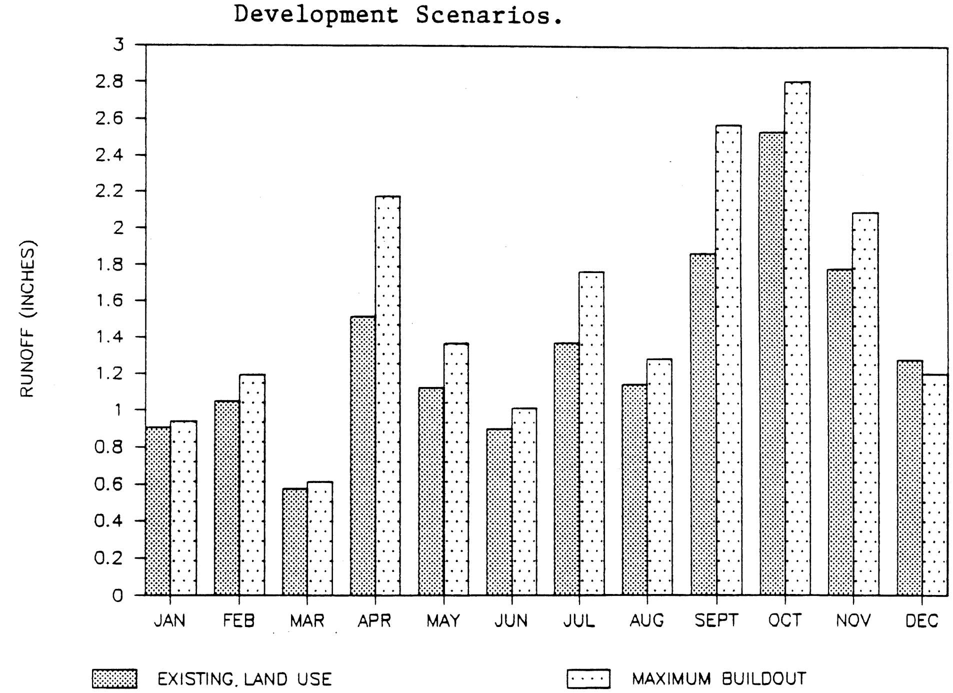

Figure V-18. "Typical" Freshwater Runoff Volumes From the Study Area Under Existing Land Use and the Maximum Buildout Development Scenarios.

The waste load allocation methodology for the Indian River, as part of the Brevard County 208 Areawide Waste Treatment Management Plan, was based on maintaining average annual concentrations of nutrients, rather than defining a critical month or storm event (Brevard County 208 Plan, 1979). The reasons for justifying such an approach were (1) severe data limitations, (2) the conservative nature of the pollutants, and (3) the assimilative characteristics of the lagoon.

A point of interest generated from this simulation experiment is the difference between the average annual concentration and the high flow concentration, emphasizing the importance of specifying a design duration of steady-state conditions for wasteload allocation studies. A wasteload allocation study using average annual concentration could significantly underestimate the impact from non-point source loads.

CHAPTER VI

SIMULATION OF WATERSHED CONTROL SYSTEM PERFORMANCE

The channel retrofitting with wetland routing strategy (watershed control system) has been selected as the best control alternative based on preliminary screening (Chapter III). In the State of Florida, emphasis is shifting to comprehensive stormwater management throughout the watershed and the use of isolated wetlands and multipurpose wet detention regional facilities to provide more natural stormwater management (Livingston, 1986). Such an approach also takes advantage of economies of scale and lower operation and maintenance costs.

By adding a control structure at the outfall of the main trunk channel of a catchment, the channel effectively becomes a wet detention basin. Wet detention basins have long been used for primary treatment (sedimentation) of wastewater and are considered proven technology. The U.S. EPA (1988) and Hyde et al. 1984) have established that wetlands under certain scenarios can improve water quality to a secondary or tertiary treatment level. Specific to an oligohaline coastal marsh at KSC receiving secondarily treated domestic wastewater, Mion (1986) demonstrated removal efficiencies of 41-98% for nitrogen and phosphorous compounds and 64-99% for copper and zinc. However, the rate and pollutant flux in a wetland varies greatly and depends on a number of factors such as the type of wetland, the application rate, hydrology, soils, and others. Therefore, the pollutant removal efficiency of a wetland treatment system has a relatively high degree of uncertainty. The long term effectiveness of such systems is unknown. Most research has been conducted on the short term scale. One of the more favorable aspects of this option is the copious area of a variety of wetlands (disturbed, undisturbed, freshwater, saltwater, etc.) at KSC. The analysis approach was intended to apply to other KSC catchments with proper definition. In summary, the wetland routing portion of the strategy is controversial and a nontraditional control alternative for stormwater management that holds great promise. Even before system performance analysis, wetlands must be recognized as the weak link in the strategy.

Table VI-1 summarizes objectives and measures of performance specific to the selected strategy. Various measures of performance are required to evaluate the system. One practical approach to optimizing complex water resources systems is through simulation models. Hydrologic, hydraulic, and treatment analyses were conducted using SWMM; an attempt to link these results with economic evaluation was also performed. The analysis was not a

Table VI-1. Multiobjective Performance Measures for Watershed Control System.

OBJECTIVE	PERFORMANCE MEASURE
Protect facilities from Standard Project Flood.	Prevent road flooding at 3 selected locations.
Do not exceed wetland assimilative capacity or change hydroperiod.	Hydraulic loading rate, (2-6 in./wk.).
Maintain aerobic sediments in channel.	Minimize water depth (not to exceed 6 ft.)
Protect wetland from excessive sedimentation.	TSS loading rate (avg. annual conc. of 5 mg/L)
Reduce "colored" ground-water discharge.	Predicted annual groundwater discharge.
Maintain estuary salinity.	No. of days in excess of 5 cfs. or cumulative discharge.
Reduce pollutant and sediment loading to the estuary.	Event Mean Concentration of (DER goal = 80% reduction).
Prevent mosquito infestation.	Minimum water depth (0.5 ft.), number of days with no flow.
Enhance channel littoral zone as natural habitat and treatment system.	Annual water level frequency.

specific design for the study area, but rather a feasibility analysis. Only sub-catchments 4 and 24 were included in this demonstration. More detailed surveying would likely be required, particularly of the wetland, before actual design and implementation. Even though continuous simulation is somewhat more sophisticated than the traditional approach of design for a single design event, it is still based on static set points for control structures. Faced with the facts that (1) all of the environmental systems being modeled are not completely understood scientifically, (2) the degree of uncertainty in the application of these models, and (3) the model results are only truly applicable for a given time series, dynamic designs are the only method available to optimize the non-steady state condition of watershed control system operation. Therefore, adjustable control structures supported by performance/monitoring data will be required to experiment and "fine-tune" the proposed system.

Preliminary Design for Demonstration Scale Project

After analysis under various configurations, the channel was found to be best retrofitted by constructing the following three structures:

1.) Emergency outfall weir near outfall of the channel (invert elevation = +4.5 ft. MSL).

2.) Side spill weir to divert flow to wetland (invert elevation = +3.0 ft. MSL).

3.) Flap-gated culverts for discharge from wetland to Buck Creek (invert elevation = +2.5 ft. MSL).

A schematic of the plan is shown in Figure VI-1. A schematic comparison of the proposed watershed control system and the existing drainage network is shown in Figure VI-2. For ease of simulation, a rectangular weir was used to control outflow from the wetland. In actual application, flap-gated culverts will prevent backflow into the system during storm surge. Although the surface elevation of the wetland bottom is estimated at +2.0 ft. MSL, it is assumed that the shoreline dike crest elevation is +5.0 ft. MSL and prevents storm surge overtopping.

Flood Control Performance

The Merritt Island area of Florida is subject to flooding from hurricanes and tropical storms and has also experienced some high waters during "Nor'easter" storms (National Oceanic and Atmospheric Administration, 1971). Factors contributing to extreme flood events are (1) storm surge, (2) water level rise due to rainfall/runoff, and (3) water setup due to wind and waves. For the initial construction of KSC facilities, the U.S. Army Corps of

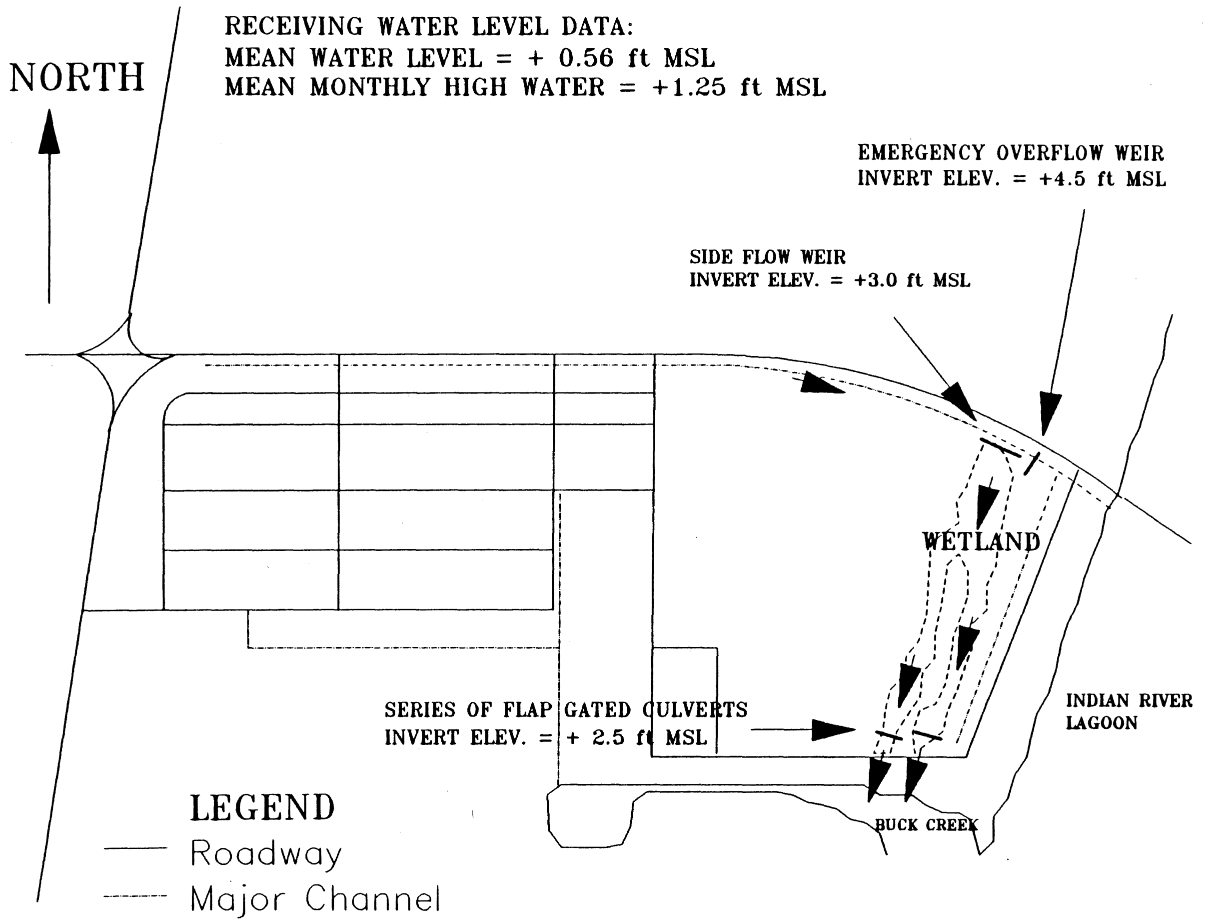

Figure VI-1. Schematic Plan for the Proposed Demonstration Scale Watershed Control System.

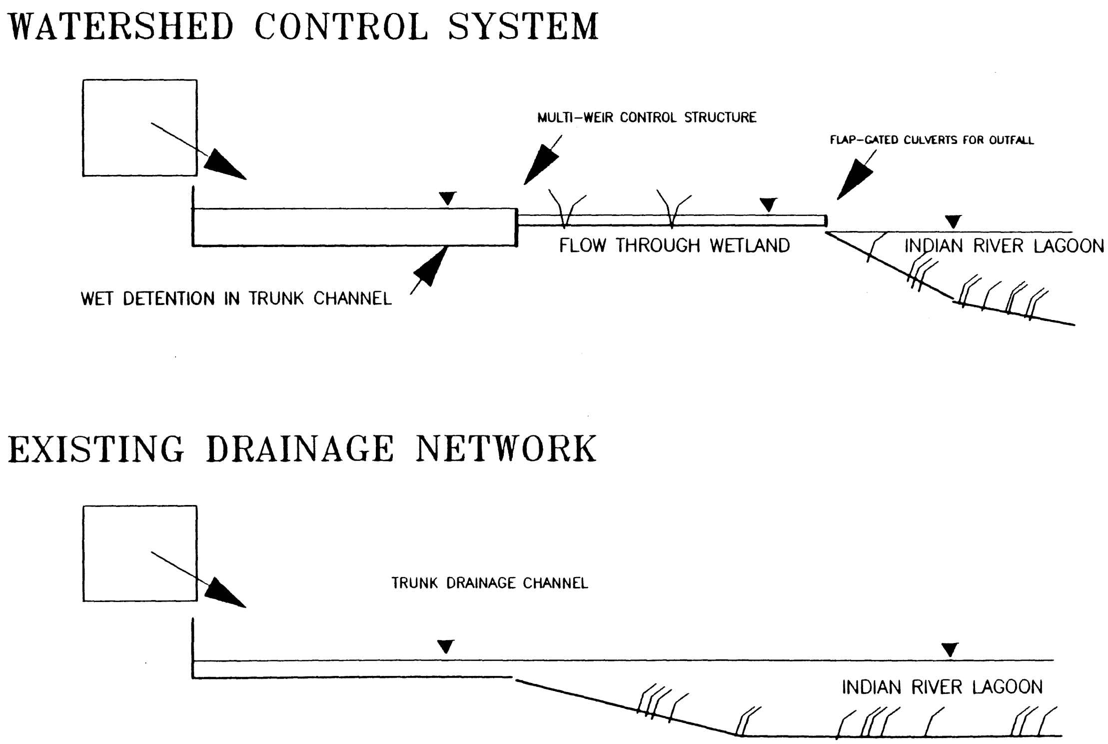

Figure VI-2. Schematic Comparison of the Existing Drainage Network and the Proposed Watershed Control System.

Engineers (COE) (1962) projected the following flood elevations:

100-yr. hurricane and wind setup	=	+7.0 ft. MSL
Standard Project Flood stage	=	+3.8 ft. MSL
Mean Lagoon water stage	=	+0.8 ft. MSL

For the Brevard County Flood Insurance study, NOAA set the 100-yr. flood elevation at +4.0 ft. MSL. This estimate was based on empirical data and high water marks recovered by the COE.

The Standard Project Flood (SPF) stage was selected for this flood analysis. The 100-yr. hurricane combined with maximum wind setup would inundate the entire drainage network as well as many of the access roads. The combined probability of this event occurring is not known. The COE study, which estimated critical wind and surge conditions near KSC, was based on statistical predictions of hurricane path, intensity and frequency, using data from past hurricanes. Considerable uncertainty is involved in these computations, especially in view of the difficulty in determining the frequency of a hurricane of a given magnitude striking the area. Therefore, as a qualification of analyses associated with this stormwater research, any appraisal of the need for flood protection to facilities is beyond the scope of this study.

Traditionally, intensity-duration-frequency (IDF) curves are used to select the SPF design storm (100-yr., 24-hr. storm). The synthetic design storm volume is then distributed over the 24 hour period according to the Soil Conservation Service (SCS) Type II distribution (for Florida conditions). An alternative, less traditional technique, is to use continuous simulation of historical storm events and then performing a frequency analysis of synthetic runoff events (the parameter of interest) rather than on precipitation events. Such procedures tend to generate more realistic designs and generally result in a lesser margin of safety. However, to follow engineering tradition (conservative) so that the flood analysis is not the center of debate (but rather the water quality control issue), the IDF procedure was used for this analysis.

Performance of the channel system under the SPF was examined for existing and retrofitted (weir) conditions under both existing development and estimated maximum buildout. Three road crossings were used as control points as measures of system performance. For this study, failure was defined as inundation of the selected road crossings for any length of time. The 100 yr.-24 hr. storm event is 10.56 inches (Florida Department of Transportation, 1987). Because the most critical runoff conditions occur in the hurricane season, typical antecedent wet season water level elevations in the channel and wetland were determined by a four month rainfall-runoff-routing simulation. Starting at the driest period of the year (May), May 1, 1951 to August 31, 1951 (the typical year) hourly rainfall data were input to RUNOFF under existing and maximum buildout conditions. The synthetic runoff generated data were inputted to the Extended Transport Block of SWMM (EXTRAN) (Roesner et al. 1989) and routed through the existing and retrofitted system.

EXTRAN is a dynamic flow model that routes inflow hydrographs through open channel and/or closed conduit system, computing the time history of flows and heads throughout the system. The program solves the full dynamic equations for gradually varied flow (St. Venant equations) using an explicit solution technique to step forward in time. The conceptual representation of the drainage network is based on the link-node concept (Roesner et al. 1989).

Resulting water levels in the system on "August 31, 1951", represented antecedent conditions for the SPF storm event. The SPF design rainfall was then inputted into the "hot started" simulator. The Indian River Lagoon stage elevation under storm surge was assumed to be constant and to have reached a peak prior to the SPF runoff event. For unretrofitted channel conditions, the storm surge was allowed to inundate the channels prior to the critical runoff period. The timing of the storm surge rise in relation to the critical runoff event can be significant; however, Dendrou and Cave (1987) found that flooding of coastal catchments in Virginia was relatively insensitive to the timing of the tide. Using a constant tidal boundary also eases analysis and adds further margins of safety. A schematic of the simulator setup is shown in Figure VI-3. Note that the wetland system was simulated as a wide shallow trapezoidal channel because of a lack of survey information. Junction and weir data input to the simulator are summarized in Table VI-2

TSS Removal Performance in Channel

The use of sedimentation basins and detention basins is a traditional engineering approach to water pollution control and peak flow reduction. The theory of storage/treatment devices has been thoroughly covered by other (Medina et al., 1981a, 1981b; Goforth, 1981; Nix, 1982; and Nix, 1985) as well as the optimization of their configurations (Nix et al., 1988 and Nix and Heaney 1988). McCuen and Moglen (1988) demonstrated how basin geometry can be manipulated to meet multiple objectives in water resources management. Many authors have documented the treatment efficiencies of various basin configurations with field studies. Unfortunately, few have used their data to calibrate design methods or deterministic models. For example, when evaluating a wet detention basin in Central Florida, Martin (1988) suggested estimates of removal efficiencies for a number of environmental factors.

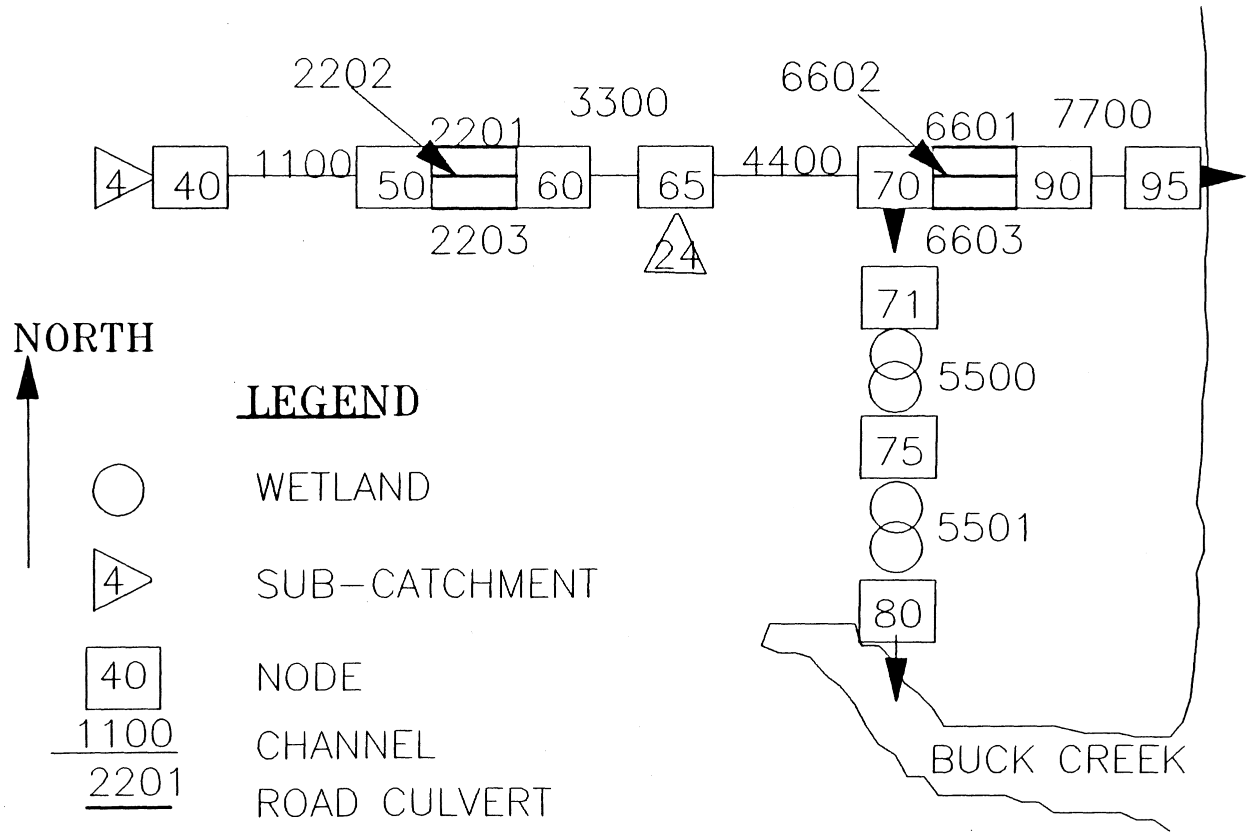

FIGURE VI-3. Schematic of EXTRAN Simulation for the Standard Project Flood Analysis.

Table VI-2. EXTRAN Junction Data for Standard Project Flood Analysis.

Existing Channel, No Weir:

JUNCTION NUMBER	GROUND ELEV. (ft MSL)	CROWN ELEV. (ft MSL)	INVERT ELEV. (ft MSL)	Qinitial (cfs)	INITIAL DEPTH (ft)
40	7.7	6.17	2.1	0.0	1.90
50	7.7	6.00	2.0	0.0	2.00
60	7.7	6.10	1.6	0.0	2.40
65	7.5	5.50	1.0	0.0	3.00
70	5.0	3.80	-0.7	0.0	4.70
90	4.1	4.80	-0.7	0.0	4.70
95	4.1	3.80	-1.7	0.0	5.70

Channel Retrofitted with Weirs:

JUNCTION NUMBER	GROUND ELEV. (ft MSL)	CROWN ELEV. (ft MSL)	INVERT ELEV. (ft MSL)	Qinitial (cfs)	INITIAL DEPTH (ft)
40	7.7	6.17	2.17	0.0	0.88
50	7.7	6.00	2.00	0.0	1.05
60	7.7	6.10	1.60	0.0	1.45
65	7.5	6.00	1.00	0.0	2.00
70	5.0	4.50	0.00	0.0	3.05
71	4.1	3.50	2.00	0.0	1.20
75	4.1	3.50	2.00	0.0	1.20
80	4.1	3.50	2.00	0.0	1.20

Weir Data:

FROM JUNCTION	TO JUNCTION	CREST HEIGHT (ft MSL)	WEIR TOP (ft MSL)	WEIR LENGTH (ft)
70	71	3.0	5.0	50
70	OUTFALL	4.5	7.5	60
80	OUTFALL	6.2	13.2	6

Detention basins are normally categorized as one of three types: (1) plug flow, (2) completely mixed, and (3) intermediately mixed. Nearly all basins can be categorized as intermediately mixed (Nix, 1985), but it is useful to conceptualize them as one of the two extreme modes in order to take advantage of simpler mathematics. Detention basins receive non-steady flows. Theoretically, mean residence time in a steady-state basin can be described by the relationship:

$$t_r = V/Q \tag{10}$$

where t_r = residence time (hrs.)
V = basin volume (ft.^3), and
Q = hydraulic loading rate (ft.^3/hr.).

However, this simple relationship does not describe the mean residence time of a detention basin under non-steady state conditions. This does not preclude, however, the possibility that the steady state equation may approximate theory under certain circumstances (Nix, 1985). Long, rectangular basins where settling is an important removal mechanism is one case. The proposed weired channel in this study was represented as a 5500 ft. long, 3 ft. deep trapezoidal wet detention basin. The proposed side-flow weir to the wetland system had an invert elevation of +3.0 ft. MSL and a width of 50 feet. Even though the basin had a very large volume and very long travel path, nearly the entire volume of the basin was dead storage. Relatively little change in live storage occurred. Therefore, the steady-state equation may approach the theoretical approximation.

This study used a deterministic model approach (Storage/Treatment Block of SWMM) supported by site specific settleability data to evaluate the relative treatment efficiencies (sedimentation of solids) in the weired trunk channel. The S/T Block uses the modified Puls method of reservoir routing (Viessman et al., 1977) to route flows through the basin. Pollutants can be routed by one of two modes: complete mixing or plug-flow. Removal is predicted with either a user supplied removal equation containing one or more terms with state variables such as residence time and inflow concentration, or a discrete particle settling routine (in the plug-flow mode only).

The hourly runoff flows and TSS loads produced by RUNOFF were routed through the proposed storage-treatment system simulated by the S/T Block of SWMM. Outflow from the basin was determined by a broad-crested weir power equation where the weir coefficient and power coefficients were assumed to be 3.33 and 1.5, respectively. The storage basin was assumed to behave as a completely mixed reactor. Sedimentation of TSS was represented by the first-order "reaction" equation developed from site-specific settleability data (presented in Chapter IV). Although plug-flow conditions combined with typical settling velocity distribution data may be more applicable to settling of solids in a very long basin, such a system could not be simulated with the existing model. An unsuccessful attempt was made to modify the S/T Block to account for more than 100 plugs within each time step or by reducing the number of plugs accounted for by increasing the time step. No reasonable combination was obtained. Removal percentage was defined as the percent of the solids load entering the basin that "decay" there.

Groundwater Discharge to Channels

For the coordinated and planned management of both surface water and groundwater, watershed models must include detailed groundwater processes with surface water processes. This significantly increases the complexity of the modeling effort. As noted in Chapter VI, the subsurface flow subroutine in RUNOFF is fairly simple in design and contains numerous parameters to be estimated. However, the parameters are physically based and some were calibrated to provide acceptable results. By placing a weir in the trunk drainage channel, the depth of water in the channels will rise to approximately the invert elevation of the weir during dry-weather. This increase of water depth in the channel can be expected to reduce the subsurface hydraulic gradient to the channel which results in a decrease of groundwater discharge.

To estimate this reduction after constructing a weir in the channel, RUNOFF model parameters representing water depth in the channel were specified to represent the invert elevation of the outfall weir. For the purposes of groundwater discharge simulation, these water depths were assumed to remain constant in the channels. During wet-weather, however, water levels in the channels rise rapidly (more rapidly than the water table) which, in effect, minimizes groundwater discharge during wet-weather. This results in groundwater discharge being primarily a dry-weather and transition between wet and dry weather period phenomenon. Therefore, assuming a constant channel stage for groundwater discharge computations provided a reasonable simulation approach. The prolonged storage of "colored" groundwater in the channel was assumed not to reduce its color content even though a dilution affect would occur.

Other Performance Measures

The trunk channel water level elevation was assumed to remain relatively static near the outfall invert elevation. Minor fluctuations will occur during wet-weather and extended dry periods due to evaporation. However dynamic, seasonal water level variations were assumed to be all but removed by a retrofitted drainage system. Water levels could be manipulated by bleeding down the basin on a scheduled basis. The determination of a mulitiobjective water level schedule in the trunk channel was beyond the scope of this study.

No general guideline criteria for freshwater discharge to an estuary could be found for the Indian River Lagoon. Most data and impacts documented are highly site specific. As a guide in selecting a performance measure, discharge from subcatchments 4 and 24 under predevelopment conditions was simulated using RUNOFF (assuming 1% hydraulically connected impervious area). Under this scenario, no surface runoff occurred during the "typical" year; all discharge was accounted by groundwater seepage. Pre-development mean and maximum discharge were 0.73 and 1.58 cfs, respectively. In order to compare annual runoff event statistics from various development scenarios, a common minimum interevent time (MIT) of 2.0 hours with a baseflow of 1.5 cfs was used to define runoff events from the hourly time series. Assuming that the time between runoff events (delta t) is exponentially distributed, the adequate MIT is the one leading to a coefficient of variation equal to one for delta t (Hydroscience, Inc., 1979).

Using common event definitions for the three development scenarios, the coefficient of variation ranged from 1.1 to 1.3. Distinct characteristic differences in the runoff time series between the development scenarios accounts for the differing coefficients of variation. The frequency of mean event flow rates (total event discharge/event duration) for each development scenario is shown in Figure VI-4. Using this figure, a mean event freshwater discharge of 5 cfs was selected as a starting point performance measure for maintenance of the Indian River Lagoon salinity. Although 5 cfs is in excess of events experienced under pre-development, it represents a compromise in reaching a pragmatic goal. Performance of the control system was determined by inputting the hourly output from the wetland outfall (represented as a second detention unit in the S/T Block) into the SWMM Statistics Block and summarizing events based on the common event definition.

Performance Measures Specific to a Wetland System in Florida

The State of Florida Henderson Wetlands Act authorizes the use of certain wetlands for the treatment of stormwater and domestic wastewater. The recent Reclaimed Water to Wetlands Rule provides design criteria for the discharge to treatment or receiving wetlands. This rule requires pretreatment for stormwater prior to discharge to the wetland so as to the protect the ecological balance and function of the wetland to be used. Efforts were made to compare performance with design criteria with the most notable exceptions being the annual average loading concentration of Total Nitrogen (as N), Total Phosphorus

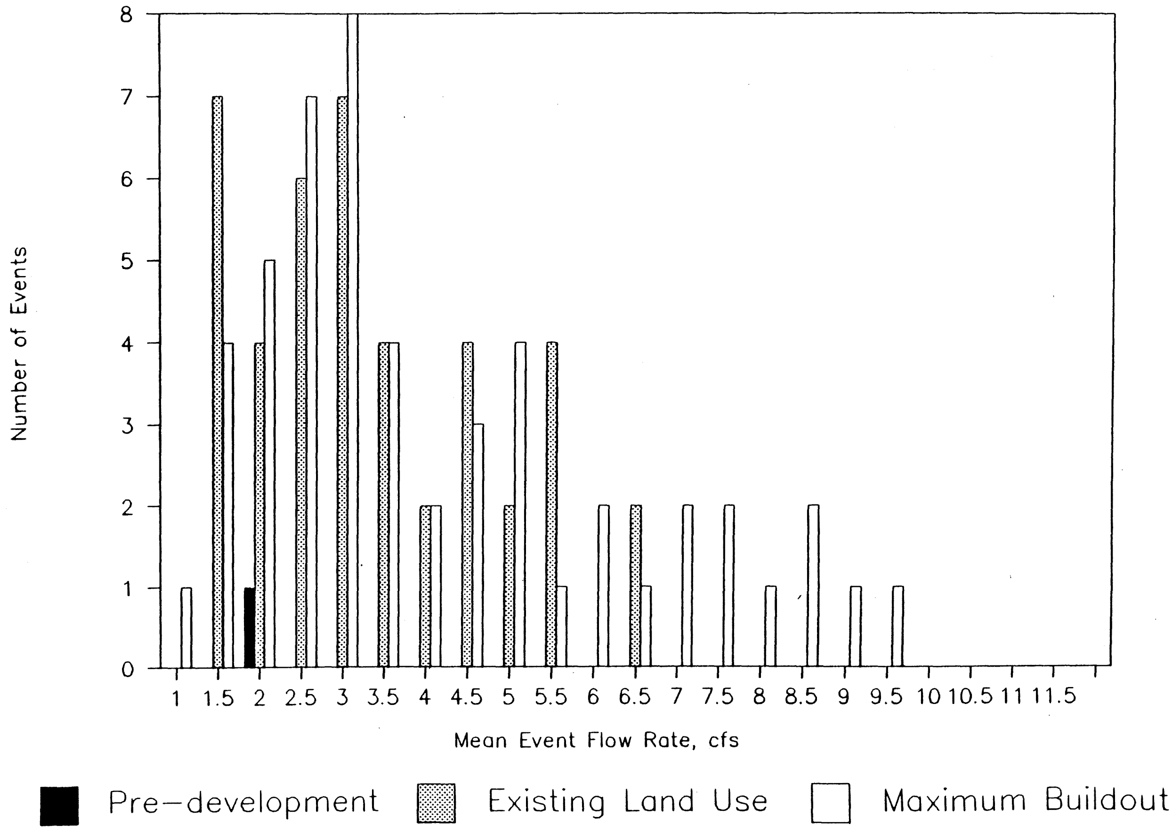

Figure VI-4. Frequency of Mean Event Discharge for the "Typical" Year Under Varying Development Scenarios.

(as P), and Carbonaceous Biochemical Oxygen Demand (CBOD5). A comparison of the Ryan and Goetzfried (1988) and Jones (1986) data on nutrient concentrations in KSC drainage channels with design criteria suggests that the criteria can be met, with perhaps the exception of TSS. This comparison is shown in Table VI-3.

Performance Results

The multiobjective performance of the system is summarized in Table VI-4. The results suggest that both the existing and retrofitted systems perform nominally (serious road crossing inundation did not occur) under SPF conditions. Although the wetland overflows under all flood scenarios, this is not considered a failure. These results suggest that additional flood protection is gained by a weir in the channel near the outfall as it prevents storm surge from entering the drainage network resulting in additional storage capacity during critical runoff periods. The wetland also provides additional storage. Increasing the Manning's n friction coefficient of the channel to represent those typical of heavily vegetated channel (Manning's n of 0.1), resulted in system failure under all scenarios. Therefore, the system must be continually maintained to perform under SPF conditions. It is also important to recognize however, that rainfall data with an hourly time step may under predict peak flows.

Table VI-3. Comparison of Untreated Drainage Water Quality with Wetland Design Criteria.

Parameter	Design Criteria	Ryan & Goetzfried (1988)(1)	Jones (1986)(2)
TKN	3 mg/L	1.3 mg/L	0.6-4.3 mg/L
TP	1 mg/L	0.27 mg/L	0.09-1.16 mg/L
TSS	5 mg/L	11 mg/L	4-398 mg/L

(1) Mean for dry weather data at station 5 near outfall.

(2) Wet-weather event composite concentration range.

Table VI-4. Comparison of Performance for Existing System vs. Retrofitted System.

PERFORMANCE MEASURES	EXISTING SYSTEM		RETROFITTED SYSTEM	
	EXISTING LAND USE	MAXIMUM BUILDOUT	EXISTING LAND USE	MAXIMUM BUILDOUT
Minimum Depth Below Road Crossings Under Standard Project Flood	1.70 ft.	0.00 ft.(1)	2.08 ft.	1.77 ft
Mean Water Depth in Trunk Channel	0.5 to 1.5 ft.	0.5 to 1.5 ft.	3.0 ft.	3.0 ft.
Annual Discharge of Groundwater	588 ac.-feet	555 ac.-feet	535 ac.-feet	516 ac.-feet
Annual TSS Load	272,000 lbs.	581,000 lbs.	37,483 lbs.(2)	121,260 lbs.(3)
Number of Freshwater Discharge Events	42	52	36	39
Number of Mean Freshwater Discharge Events in Excess of 5 cfs.	6	14	1	8

(1) Road flooding had a duration of one hour.

(2) 86 % removal efficiency.

(3) 79 % removal efficiency.

Even though the simulated system meets flood control criteria, there is uncertainty in some assumptions made. For instance, what will occur when the wetland overflows has not been documented. There are parcels of undeveloped lowlands in that part of the catchment; however, without survey information, the storage capacity is not known. In addition, it was assumed that the shoreline dike in the catchment is continuous, in excess of +4.0 ft. MSL in elevation, and will prevent storm surge from inundating the lowland areas of the catchment. Perhaps most importantly is the question of whether the use of the IDF derived 100yr.-24hr. storm event occurring simultaneously with the 100-yr. coastal flood elevation for flood analysis (the joint probability method) is too stringent and leads to over designed flood control. Using this traditional civil engineering design approach has serious consequences in the design of multiobjective watershed control systems (i.e. the flood control criteria dominates the design). A better method would be to analyze historical runoff data, or synthetic runoff data generated from historical rainfall , and receiving water elevations; the flood control design could then be based on critical "real" storm events. This method, however, has not made it into the engineering mainstream primarily because of a lack of sufficient databases and a general lack of hydrologic analysis capability in city and facility engineering staffs.

Improvements by way of reducing groundwater discharge are poor. Compromises on flood protection and an aerobic environment would be required to improve the system. The model inaccurately represented groundwater discharge under increasing development scenarios. Rather than representing groundwater discharge as a function of channel length, discharge coefficients in the groundwater subroutine in RUNOFF were developed to represent discharge as a function of catchment area. Although this approach is not physically accurate, it is a convenient lumped parameter modeling method for calibrated catchments. Hindsight has identified methods to incorporate channel length in the discharge coefficients (see Appendix C).

Theoretically, wet detention in the weired channel provides adequate sedimentation that in effect meets The Stormwater Rule goals of 80% pollutant removal. However, the criteria for TSS loading to the wetland was not met which may lead to excessive sedimentation in the wetland. In addition, scour and resuspension in the channel and the wetland under extreme storm events was not examined.

Wetlands performance, although not optimal, could be easily adjusted to provide adequate wetland protection with additional data and/or additional wetland acres. The performance of the system with specific respect to the wetland design criteria is shown in Table VI-5. Detention time is nearly doubled by raising the outfall invert several

Table VI-5. Comparison of Performance for Existing Land Use and Maximum Buildout Specific to Florida Wetland Design Criteria.

PERFORMANCE MEASURE	DESIGN CRITERIA	EXISTING LAND USE	MAXIMUM BUILDOUT
Mean Annual Average TSS Concentration	5 mg/L	16 mg/L	39 mg/L
Annual Average Hydraulic Loading Rate	2 - 6 in./wk.	5.1 in./wk.	6.5 in./wk.
Minimum Detention Time (annual basis)	14 days	6 days	5 days

inches; this tradeoff needs to be examined in reducing flow through the wetland (i.e. primarily mosquito control) and potential adverse backwater effects in the system. In addition, the number of high freshwater discharge events and TSS loads are significantly reduced. The relative effectiveness in reducing mean freshwater discharge events for existing land use and maximum buildout is illustrated in Figures VI-5 and VI-6, respectively.

Cost Analysis

Without performing a complicated and likely controversial benefit assessment, it was not possible to quantify the "benefits" and "costs" of a watershed control system and comparing it to current project specific stormwater management practices at KSC. However to put the proposed watershed control program into perspective, cost analysis was required.

It would be incorrect to directly compare single objective, site specific project costs with multiobjective watershed project costs. However, from an operational decision making standpoint, it is possible to weigh the current single objective regulatory requirement costs with an experimental, process-control oriented system that may take advantage of economies of scale and more efficient O&M but require more manpower to perform well. It is also

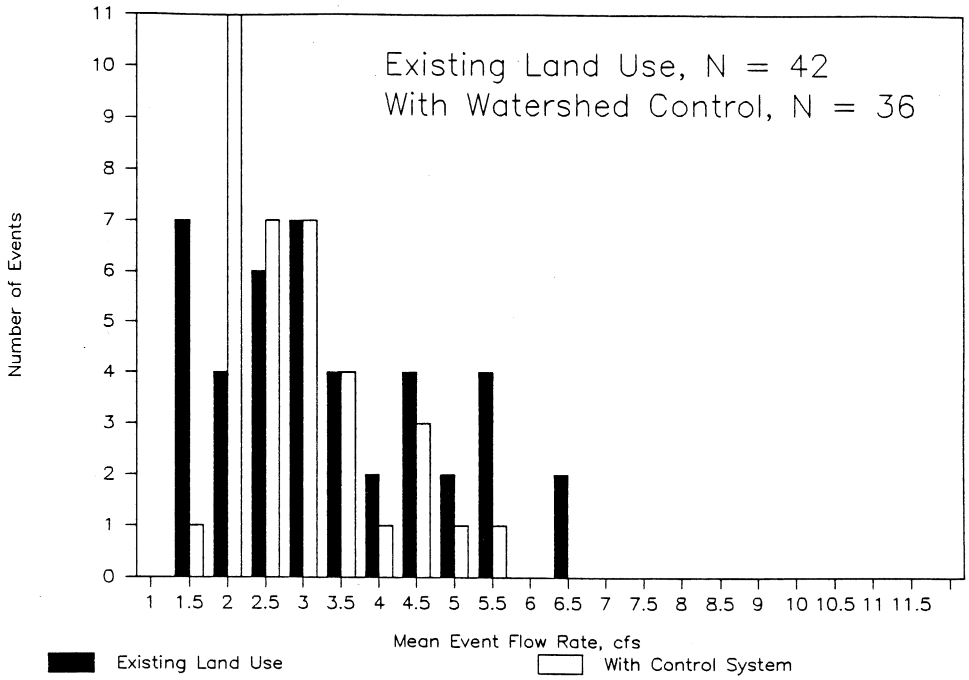

Figure VI-5. Frequency of Mean Event Discharge for Non-Control and Watershed Control Under the Existing Land Use Scenario.

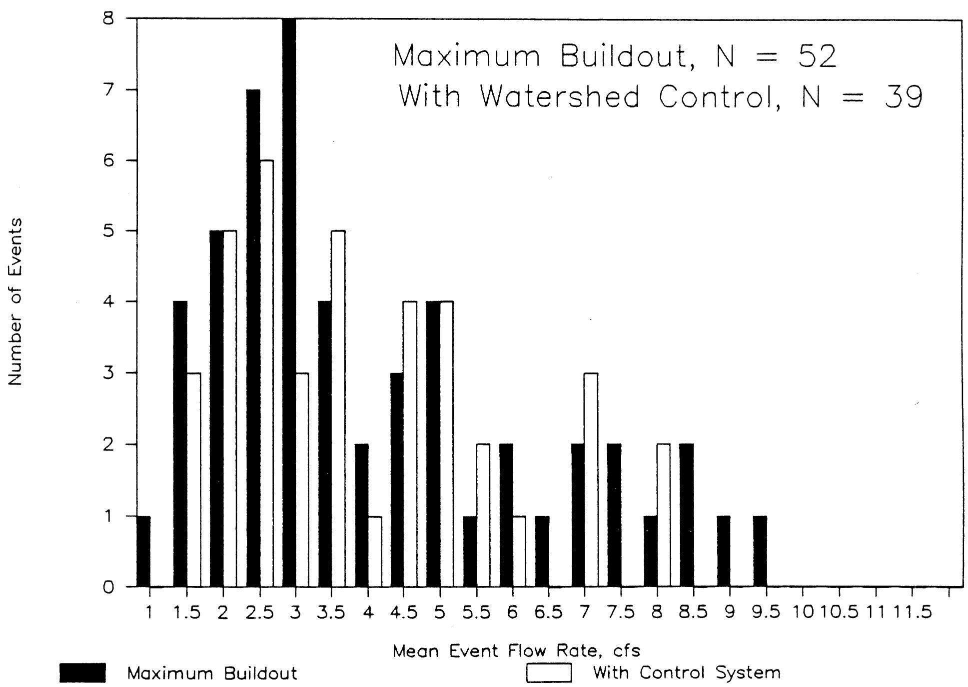

Figure VI-6. Frequency of Mean Event Discharge for Non-Control and Watershed Control Under the Maximum Buildout Scenario.

important to recognize the difficulty in estimating the number and size of single project basins for the future in a proposed retrofitted catchment. Several subcatchments may have to be retrofitted to support a centralized area of development. For example, the Industrial Area would require retrofitting projects for three subcatchments. The KSC VAB area might require retrofitting of as many as four subcatchment outfalls. Therefore this cost analysis only represents a typical cost of retrofitting a single "typical" subcatchment.

The costs in 1989 dollars for the different management techniques were calculated by estimates provided in the literature and derived from interviews and multiplying them by appropriate historical cost factors provided by the 1989 Means Cost System. Cost estimates are shown in Tables VI-6 and VI-7. The typical costs of constructing control structures for a weired trunk channel with flow through wetland system was $28,100.00 in 1989 dollars. Assuming process control and monitoring will be required for the life of the project, average annual costs were approximated at $17,100.00/yr. in 1989 dollars.

Table VI-6. Cost Analysis for Demonstration Scale Watershed Control System.

Surveying	
Man Hours	64 hr.
Hourly Wage	13 $/hr.
Overhead Multiplier	3
Cost of Surveying	2496.00 $
Design Engineering	
Engineer Man Hours	160 hr.
Technician Man Hou	40 hr.
Engineer Wage	20 $/hr.
Technician Wage	10 $/hr.
Overhead Multiplie	3
Cost of Design Engineering	10800.00 $
Permitting	
Permit Specialist Man Hours	40 hr.
Permit Specialist Wage	12 $/hr.
Overhead Multiplier	1.2
Cost of Permitting	576.00 $
Control Structure Construction	
Number of 40,' 30" Corrugated Aluminum Culverts w/ Flap Gates and Adjustable Flow Restrictors	4
Cost of Culvert and Installation	1300 $/culvert
Emergency Overflow Weir Construction	5000 $
Side Flow Weir to Wetlands Construction (adjustable)	4000 $
Cost of Control Structure Construction	14200.00 $

Table VI-6--continued.

Monitoring	
Number of Monitoring Stations	5
Number of Years of Monitoring	10 yrs.
Analytical Cost per tation	1500 $/st/yr.
Sampling Man Hours	36 hrs/yr.
Sampling Technician	8 $/hr.
Cost of Monitoring	77880.00 $/10 yrs.
Process Control	
Number of Years of Controlling	10 yrs.
Controller Man Hours	560 hrs./yr.
Controller Wage	15 $/hr.
Cost of Process Control	84000.00 $/10 yrs.
Additional Operation & Maintenance	
Number of Years of O&M	10 yrs.
Number of Pieces of Heavy Equipment	2 equip.
Heavy Equipment Operation Cost	1.25 $/hr./equ.
Crew Size	2 men
Hourly Wage	10 $/hr.
Time per Man per Year	40 hrs./yr.
Cost of Additional Operation & Maintenance	9000.00 $/10 yrs.

****
Estimated Total Project Cost Over 10 Years: 198952 $/10 yrs.
****

Table VI-7. Cost Analysis for Single Project Basins in a Subcatchment.

Design Engineering	
Engineer Man Hours per Basin	30 hr.
Technician Man Hours per Basin	8 hr.
Engineer Wage	20 $/hr.
Technician Wage	10 $/hr.
Overhead Multiplier	3
Cost of Design Engineering	2040.00 $/Basin
Permitting	
Permit Specialist Man Hours per Basin	30 hr.
Permit Specialist Wage	12 $/hr.
Overhead Multiplier	1.2
Cost of Permitting	432.00 $/Basin
Retention/Detention Basin Construction	
Average Construction Cost per Basi	30000 $/Basin
Cost of Basin Construction per Year	30000 $/Basin
Additional Operation & Maintenance	
Number of Years of O&M	10 yrs.
Number of Pieces of Heavy Equipment	2 equip.
Heavy Equipment Operation Cost	1.25 $/hr/equ.
Crew Size	2 men
Hourly Wage	10 $/hr.
Time per Man per Year	64 hrs./yr.
Cost of Additional Operation & Maintenance	1440.00 $/yr.
Fixed Cost per Basin	32472 $/Basin
No. of Basins Constructed in 10 Year Period	5
Annual Cost	1440 $/yr.

****
Estimated Total Project Cost Over 10 Years: 176760 $/10 yrs.
****

CHAPTER VII

SUMMARY AND CONCLUSIONS

Summary

Evidence documenting the degradation of the Indian River Lagoon is largely circumstantial. Even so, land drainage has been identified as a primary factor in the degradation. Agencies are now considering alternatives to mitigating these effects such as retrofitting existing drainage networks in problem areas and implementing a watershed approach to stormwater management. These water management problems in the Indian River Lagoon are complex. Rather than identify additional environmental problems associated with land drainage in the Indian River Lagoon basin, this study attempted to evaluate mitigation alternatives for the problems already identified. It examined the fundamental premise of stormwater management, the multiple criteria required for a watershed control system, and attempted to use simulation experiments to test various objective concerns and make recommendations for practical application. In summary, it examined, in detail, critical technical questions regarding the catchment hydrology, channel hydraulics, and the treatment potential of the existing drainage system.

The results of this study include:

1) A literature review of receiving water effects near KSC.
2) The identification of issues and philosophies related to water resources management at KSC.
3) Identification of watershed management objectives and those competing in the Indian River Lagoon Basin near Merritt Island.
4) Critique of previous watershed modeling efforts conducted in the Indian River Lagoon Basin.
5) Collection of site specific hydrologic data for model calibration.
6) Calibration of SWMM, and with it, demonstrated cause and effect relationships in the generation of runoff and TSS loads to the receiving water.
7) Examined the conjunctive use application of modeling with SWMM.
8) Preliminary screening of water management approaches for KSC-wide application.
9) Complete description of the physical layout of a proposed watershed control system.

10) Simulation of the relative effectiveness of the watershed control system.

11) A cost analysis of the proposed watershed system versus the current stormwater management approach.

Negative contributions to the Indian River Lagoon that are associated with land drainage runoff include nutrients, muck, "colored" groundwater, excessive freshwater, suspended solids, and various toxins and pathogens. In an effort to address these problems, the use of watershed models are expected to increase significantly.

Environmental management perspectives of stormwater runoff range from risk aversion to cost-effective approaches (cost-benefit). While risk aversion is economically infeasible, the cost-benefit approach is difficult to apply due to the lack of data on benefits. Florida is considered a leader in stormwater management regulation in the nation. Its Stormwater Rule is based on static controls to achieve cost-effective performance. Several problems remain (e.g. piecemeal approach can lead to combined effects, lack of long-term performance data, poor system performance in Florida Flatwood areas, and a lack of retrofitting of existing drainage networks).

The study area was a typical "developed" catchment at KSC that is 1900 acres in area. It can be described as a

Florida Flatwood watershed that has been "improved" with an extensive drainage network of open channels that were designed to pass the Standard Project Flood. It has two major outfalls to the Banana River portion (Planning Segment B2) of the Indian River Lagoon. Approximately 350 acres or 18% of the catchment is impervious. The depth to the water table ranges from 2.5 to 4.5 feet below the land surface. Segment B2 is not influenced by astronomical tide but does undergo quasi-periodic fluctuation due to aeolian tide or wind setup. This phenomenon causes reversible flow and backwater effects in the channels near the outfalls during dry weather.

Objectives of an ideal control system were identified as (1) flood protection, (2) reduce groundwater discharges, (3) reduce pollutant loads, (4) maintain an aerobic environment in the channel waters and sediments, (5) reduce extreme freshwater discharge events, (6) enhance the littoral habitat of the channels, and (7) prevent mosquito infestation. Measures of performance for each objective were then developed based on regulatory guidelines and literature suggestions. Using a simple decision-aiding matrix (primarily based on the Cardinal Utility), nine potential control systems were screened. The option of channel retrofitting with wetland routing was selected as most likely to provide the best overall system performance.

The study area lacked an appropriate database; therefore, ten months of site-specific data on precipitation, water stage, water table elevation, channel water velocity, event mean concentration of TSS loads, and TSS settleability were collected. In addition, urban and drainage features were added to the KSC GIS. With these data, the cause and effect relationships of the catchment hydrology, channel hydraulics, and pollutant loads were documented and summarized using the calibration and verification of the SWMM model. The simulation included groundwater discharge to the channels. The surrogate pollutant, TSS, was best represented by a rating curve to describe scour in the drainage channels. The generation of runoff was most sensitive to the depth to the water table, catchment width, and percent imperviousness.

This calibrated model was then used to estimate "typical" loadings to the estuary. This was performed by running a continuous simulation of the RUNOFF block of SWMM with a "typical" year (1951) of hourly rainfall data. For the estimates it was assumed that no TSS removal occurred in the trunk channels. Yearly loadings were estimated for both existing land use and the maximum buildout scenario. The model estimated that annual freshwater discharge would increase by 18% and annual TSS loads increase by 290%.

The calibrated model was then used to evaluate the relative effectiveness of the proposed watershed control system under existing land use and maximum buildout. The feasibility of the proposed system was evaluated in detail for only one of the outfalls of the study area. The proposed demonstration scale project involves constructing three adjustable water control structures and connecting a partially disturbed interdunal swale freshwater wetland site to the drainage network. Two weir control structures are proposed near the outfall of the trunk channel; one functions as an emergency spillway discharging directly to the estuary and the other, a side-spill weir, routes dry and wet-weather flows through the forty acre wetland site. Discharge from the wetland will be controlled by a series of adjustable, flap-gated culverts. The trunk channel will provide primary sedimentation as a wet detention basin and the expected function of the wetland system is to provide nutrient assimilation, additional storage, and enhanced evapotranspiration.

The system was then evaluated using the general objectives and objectives specific to the wetland system established by the regulatory agencies. Even though the system was not optimized due to analytical uncertainty, it was apparent from the simulation experiments that the system could not satisfy all of the objectives. Without sufficient scientific evidence documenting the importance of individual factors contributing to the degradation of the Indian River Lagoon, it would be erroneous to prioritize the various water quality improvement objectives. With SPF based flood control as a pre-emptive goal, the simulated system only achieved nominal performance for flood control, TSS load reduction, and maximum water depth. Long-term performance of the wetland system could not be predicted; however, it will likely result in the reduction of dissolved water quality constituents.

The simulation experiments clearly illustrated that the system will have great difficulty in meeting the groundwater discharge and water level fluctuation criteria under all development scenarios. Without periodic drawdown, water levels in the system will be near static. Groundwater discharge was estimated to be reduced only 9%. Under maximum buildout, sedimentation in the wetland may be a problem. Improvements in reducing the number of freshwater discharge events in excess of the criteria are made; however, numerous failures were recorded under maximum buildout. To improve overall performance, this work suggests that further compromises in the objectives will be required as well as collecting actual system performance data. This will involve critiquing the scientific basis of the criteria, in particular the justification of using synthetic design storms for flood control design and the measurement of actual TSS loads to the wetland.

An itemized cost analysis determined that the fixed cost of the proposed system would be approximately $28,072.00 in 1989 dollars. Annual process control and monitoring costs were estimated to cost $17,088.00 per year. Over a ten year life of the project, the cost would be expected to be $198,953.00 in 1989 dollars. Based on construction trends since 1984, it was estimated that the construction of individual project stormwater basins for the demonstration subcatchment would cost $176,760.00 over ten years in 1989 dollars. The cost of constructing a typical retention basin was estimated from previous KSC construction to be approximately $30,000.00. Although it would be incorrect to directly compare single objective, site specific project costs with multiobjective watershed control project costs, such a comparison illustrates that the proposed system would not only be expected to be more effective in addressing receiving water quality issues but cost competitive as well.

Conclusions

To address receiving water effects in the Indian River Lagoon, stormwater management on the watershed scale must take on a multiobjective analysis approach. As this study highlights, addressing the multiobjective criteria required to design a watershed control system is no trivial endeavor. From a practical standpoint, all of the criteria can not be optimally achieved which raises some very interesting tradeoff questions. For example, can a portion of the flood protection margin of safety be traded for expected improvements of water quality? Do the expected benefits of water quality improvements due to an untested flow-through wetland treatment system compensate for utilizing the wetland resource? Although costs would be expected to be comparable to current management practices, performance would certainly be enhanced. Optimization and justification of such systems can not be made until adequate receiving water analyses are conducted.

The flood protection performance of the system is very sensitive to the height of vegetation in the channels. This study suggests that channels must be maintained regularly. However, data from this study also support the hypothesis that the regular denudation of vegetation in the channel results in a significant contribution to the TSS load from the catchment. Where practical, it is recommended that channel vegetation be maintained by methods other than scraping and removal of sod (e.g. mowing or clipping during dry weather). Periodically the wetland system may require maintenance to reduce organic matter accumulation, remove accumulated sediment, and reduce the buildup of hillocks which encourage channelization of flow, Case studies show that burning, water level drawdown, and scraping are commonly used for wetland maintenance. By reducing TSS production due to channel scour, wetland sedimentation is not expected to be a problem. Therefore, periodic burning and water level drawdown may prove to be sufficient wetland maintenance. This subject is covered in more detail by others (Hyde et al. 1984; USEPA, 1988).

A number of recommendations can be made with respect to the application of SWMM to additional watershed analysis in the Indian River Lagoon Basin. A variable time step option is available in RUNOFF (found in the B3 card); however, other modules of SWMM do not have such an option which leads to problems in coupling RUNOFF with other SWMM modules (e.g. large continuity differences between the modules). A constant time step is recommended if RUNOFF output is to be interfaced with other SWMM modules. The groundwater subroutine in SWMM is simple yet semi-physically based. For future applications to surface-subsurface water management it is recommended that groundwater discharge coefficients be developed to represent seepage face length (channel length) rather than catchment area. Also, as the SWMM manual notes, this subroutine does not account for exfiltration from the channel. This was found to be significant in this study, particularly during dry weather. A nice review of transient seepage can be found in Bouwer (1978). For evaluation and design, plug flow best represents the wet detention in the retrofitted channels. However without major modification of the model (S/T Block), it is not possible to continuously simulate plug flow in these lengthy channels. If model modification is avoided, one option would be to develop and utilize a historical design storm time series of pollutant load for predicting the relative treatment effectiveness of the retrofitted channel.

Although an estimate of the settleability of TSS in the stormwater effluent was provided in this study, certainly additional effluent treatability study is warranted. Information on the settleability of nutrients, metals, and hydrocarbons would help "fine tune" the proposed system. The establishment of a long-term lagoon water level data base would be beneficial to future water resources analysis and design and estuarine research. Mathematical models of seagrass systems offer an interesting new approach to receiving water analysis. The feasibility of incorporating such models into environmental management (i.e. used as an all encompassing measure of system performance) should be investigated.

Hydrologic simulation is a very powerful technique in organizing and understanding the catchment hydrology and hydraulics and in examining the relative effectiveness of management scenarios. However, this detailed analysis has highlighted that due to the analytical uncertainty, system performance can only be optimized through dynamic designs supported by dynamic simulation. As Schilling (1985) concludes in his excellent overview of real-time control of urban runoff quality management,

> "There is no need for further research into even more involved mathematical techniques, but there is a great need for...joint research/application projects and enhanced communications between scientists, engineers, and operators."

Perhaps the time has come for the implementation of experimental, dynamic watershed control systems to address the multiple water management objectives in the Indian River Lagoon Basin.

APPENDIX A

WATER LEVEL DATA BASE ASSEMBLED FOR THIS STUDY

	Installed	Recovered
1. Station SW1	February 5, 1988	November 5, 1988
2. Station SW2	March 4, 1988	November 2, 1988
3. Station SW3	March 4, 1988	November 5, 1988
4. Station SW4	January 29, 1988	November 25, 1988

APPENDIX B

LISTING OF SWMM INPUT FILES FOR THE RUNOFF BLOCK FOR THE STUDY AREA UNDER EXISTING LAND USE AND MAXIMUM BUILDOUT

```
*  KSC STORMWATER MANAGEMENT
*
* DREW B. BENNETT, UNIVERSITY OF FLORIDA
*
* INPUT FILE FOR RUNOFF BLOCK
****************************************************************************
*  CONTINUOUS SIMULATION FOR INDUSTRIAL AREA: 1951: CALC QUALITY & TSS LOAD *
****************************************************************************
*  PREDICTION OF RUNOFF AND TSS LOAD UNDER EXISTING LAND USE               *
****************************************************************************
SW  1        8       25
MM  7    9  12   13   14   15   16 17
@   9  'STMET51.OUT'
@  25  'INDRO.INT'
$RUNOFF
A1  'RUNOFF FOR INDUSTRIAL AREA'
A2  'SIX SUB-CATCMENTS, 1900.5 ACRES'
*
B1   0        0     1      0        1        1    1    0    1      1    51
B2   0        0     2
B3   900.0   3600.0   7200.0    2        8760
B4    30.0        0.01
*
D1   1
*
F1 0.09 0.12  0.18  0.22  0.24  0.22  0.21  0.21  0.21 0.19 0.17 0.11 0.08
*
G1  444    60   5       15    4000  0.00035 2 2       0.044    4      0.1
G2   0  0.04        3.3     3
G1  244    60   5       15    6000  0.00046 2 2       0.04     4      1.0
G2   0  0.0         3.3     3
G1  555    10   5       15    4000  0.0001  2 2       0.04     4      0.0
G2   0  0.04        3.3     3
G1  155    10   5       15    5600  0.0004  2 2       0.04     4      0.1
G2   0  0.04        3.3     3
G1  111    10   5       15    6200  0.0001  2 2       0.08     4      1.5
G2   0  0.0         3.3     3
G1  333    10   5       15    4000  0.0004  2 2       0.04     4      0.1
G2   0  0.04        3.3     3
*
H1 1   4   444   4000    167   18  0.00012  0.013 0.25  0.05 0.4 10  5 0.00115
H2  4       444      1       1     0.0      8.0     4.5   1.8       -1.0
H3  1.2E-4  2.0     0   1 1.2E-4  0.3  0.03 0.05   5.0       0.035
H4    10       15            0.001  0.00003   4.5
*
H1  1  24   244   7640    316    5  0.00012  0.013 0.25  0.05 0.4 10  5 0.00115
H2  24      244      1       1     0.0      5.0     3.8   0.0        2.9
H3  1.2E-4  2.0     0   1 1.2E-4  0.3  0.03 0.05   5.0       0.035
H4    10       15            0.001  0.00003   4.5
*
H1  1  5    555   3000    133  15 0.000012  0.013  0.25  0.05 0.4 10 5 0.00115
H2  5       555   1       1     0.0     8.0  4.5  4.7  -1.0
H3  1.2E-4  2.0     0   1 1.2E-4  0.3  0.03 0.05   5.0       0.035
H4    10       15            0.001  0.00003   4.5
```

```
*
H1   1  3    333   4000    489  4.5 0.000012  0.013  0.25  0.05 0.4 10 5 0.00115
H2   3       333   1       1       0.0     5.0  3.8   3.8  -1.0
H3   1.2E-4  2.0    0    1 1.2E-4  0.3  0.03 0.05   5.0        0.035
H4     10       15            0.001  0.00003   4.5
*
H1   1  1    111   6300    577  0.0 0.000012  0.013  0.25  0.05 0.4 10 5 0.00115
H2   1       111   1       1       0.0     8.0  4.5  3.0  1.5
H3   1.2E-4  2.0    0    1 1.2E-4  0.3  0.03 0.05   5.0        0.035
H4     10       15            0.001  0.00003   4.5
*
H1   1  15   155   3600    218.5 15 0.000012  0.013  0.25  0.05 0.4 10 5 0.00115
H2   15      155   1       1  0.0      8.0      4.5   1.8       -1.0
H3   1.2E-4  2.0    0    1 1.2E-4  0.3  0.03 0.05   5.0        0.035
H4     10       15            0.001  0.00003   4.5
*
J1   1  1  0  0  4.0  0.0  1.0  0.0  0.0  0.0  367
*
J2   'GENERAL'  0  0 10000  1.0  5.0   0.0  0.0  0.0
J3   'TSS'  'MG/L'  0  1  0  0  0  100000  1.0  5  0.0 0.0 1.15 40 0.0 0.0 0.0
J4   0  0  0.0
*
L1   1  1  0   1  0/
L1   3  1  0  66  0/
L1   4  1  0 138  0/
L1   5  1  0  87  0/
L1   15 1  0 102  0/
L1   24 1  0  40  0/
*
M0   1.0
M1    2      0
M2    1      0        0
M3    10 60
$ENDPROGRAM
```

```
*  KSC STORMWATER MANAGEMENT
*
* DREW B. BENNETT, UNIVERSITY OF FLORIDA
*
* INPUT FILE FOR RUNOFF BLOCK
*******************************************************************************
*  CONTINUOUS SIMULATION FOR INDUSTRIAL AREA: 1951: CALC QUALITY & TSS LOAD *
*******************************************************************************
*  PREDICTION OF RUNOFF AND TSS LOAD UNDER MAXIMUM BUILDOUT
*******************************************************************************
SW  1        8        25
MM  7     9  12    13    14    15    16 17
@   9  'STMET51.OUT'
@  25  'INDRO.INT'
$RUNOFF
A1  'RUNOFF FOR INDUSTRIAL AREA'
A2  'SIX SUB-CATCMENTS, 1900.5 ACRES'
*
B1   0       0     1      0        1       1    1    0    1      1    51
B2    0        0        2
B3   900.0   3600.0   7200.0     2       8760
B4    30.0        0.01
*
D1   1
*

F1 0.09  0.12  0.18  0.22  0.24  0.22  0.21  0.21  0.21 0.19 0.17 0.11 0.08
*
G1  444    60   5       15    4000  0.00035 2 2       0.044    4      0.1
G2   0  0.04        3.3     3
G1  244    60   5       15    6000  0.00046 2 2       0.04     4      1.0
G2   0  0.0         3.3     3
G1  555    10   5       15    4000  0.0001  2 2       0.04     4      0.0
G2   0  0.04        3.3     3
G1  155    10   5       15    5600  0.0004  2 2       0.04     4      0.1
G2   0  0.04        3.3     3
G1  111    10   5       15    9000  0.0001  2 2       0.08     4      1.5
G2   0  0.0         3.3     3
G1  333    10   5       15    5000  0.0004  2 2       0.04     4      0.1
G2   0  0.04        3.3     3
*
*
*
*
H1 1   4   444    4000     167    18  0.00012  0.013 0.25  0.05 0.4 10  5 0.00115
H2  4       444       1        1      0.0      8.0      4.5   1.8        -1.0
H3  1.2E-4  2.0    0    1 1.2E-4  0.3  0.03 0.05   5.0        0.035
H4    10        15            0.001  0.00003   4.5
*
H1  1  24   244   7640     316    18  0.00012  0.013 0.25  0.05 0.4 10  5 0.00115
H2  24      244       1        1      0.0      5.0      3.8   0.0        2.9
H3  1.2E-4  2.0    0    1 1.2E-4  0.3  0.03 0.05   5.0        0.035
H4    10        15            0.001  0.00003   4.5
```

```
*
H1  1  5    555   3000    133  18 0.000012  0.013  0.25  0.05 0.4 10 5 0.00115
H2  5       555   1       1     0.0      8.0  4.5  4.7  -1.0
H3  1.2E-4  2.0    0    1 1.2E-4  0.3  0.03 0.05   5.0        0.035
H4    10       15             0.001  0.00003   4.5
*
H1  1  3    333   4000    489   18  0.000012  0.013  0.25  0.05 0.4 10 5 0.00115
H2  3       333   1       1      0.0     5.0  3.8   3.8  -1.0
H3  1.2E-4  2.0    0    1 1.2E-4  0.3  0.03 0.05   5.0        0.035
H4    10       15             0.001  0.00003   4.5
*
H1  1  1    111   6300    577  18   0.000012  0.013  0.25  0.05 0.4 10 5 0.00115
H2  1       111   1       1      0.0     8.0  4.5  3.0  1.5
H3  1.2E-4  2.0    0    1 1.2E-4  0.3  0.03 0.05   5.0        0.035
H4    10       15             0.001  0.00003   4.5
*
H1  1  15   155   3600    218.5 18 0.000012  0.013  0.25  0.05 0.4 10 5 0.00115
H2  15      155   1       1  0.0     8.0      4.5   1.8        -1.0
H3  1.2E-4  2.0    0    1 1.2E-4  0.3  0.03 0.05   5.0        0.035
H4    10       15             0.001  0.00003   4.5
*
J1  1  1  0  0  4.0  0.0  1.0  0.0  0.0  0.0  367
J2  'GENERAL'  0  0 10000  1.0  5.0   0.0  0.0  0.0
J3  'TSS'  'MG/L'  0  1  0  0  0  100000  1.0  5  0.0 0.0 1.15 40 0.0 0.0 0.0
J4  0  0  0.0
*
*
L1  1  1  0 429  0/
L1  3  1  0 273  0/
L1  4  1  0 138  0/
L1  5  1  0  87  0/
L1  15 1  0 102  0/
L1  24 1  0 210  0/
*
M0  1.0
M1   2      0
M2   1      0        0
M3  10 60
$ENDPROGRAM
```

APPENDIX C

DEVELOPMENT OF DISCHARGE COEFFICIENTS IN SWMM RUNOFF MODULE TO REPRESENT GROUNDWATER DISCHARGE AS A FUNCTION OF CHANNEL LENGTH.

For infiltration and drainage to an adjacent channel see example in SWMM manual (Huber and Dickinson, 1988), page 555. The example develops discharge coefficients as,

$$A1 = A3 = 4K/L^2,$$

$$A2 = 0, \text{ and}$$

$$B2 = 2$$

The general groundwater discharge equation is then solved and multiplied by the catchment area. To negate the catchment area and calculate groundwater discharge as a function of channel seepage face,

$$A1 = A3 = 4K/L^2 * \frac{[\text{Seepage Face Area (ft.}^2)]}{[\text{Catchment Area (acres)}]} \quad \text{(C-1)}$$

REFERENCES

Bartleson, R.D. The Relative Influence of Current Reduction by Seagrasses on Sediment Nutrients and Seagrass Growth in High and Low Nutrient Waters: A Simulation Model and Field Observations. M.S. Thesis, Univ. of Florida, Gainesville, 1988.

Bedient, P.B. and W.C. Huber. Hydrology and Floodplain Analysis. Addison-Wesley Publishing Company, Reading, MA. 1988.

Bennett, D.B. Analysis and Prediction of Shallow Groundwater Level Behavior in Merritt Island, Florida. Unpublished, special project in environmental engineering. Univ. of Florida, Gainesville. 1988.

Black, A.P. and R.F. Christman. Characteristics of Colored Surface Waters. Jour. AWWA, Vo. 55, No. 6, 753-770, 1963.

Bouwer, H. Groundwater Hydrology. McGraw-Hill Book Company, New York. 1978.

Brevard County 208 Areawide Waste Treatment Management Plan. Brevard County Department of Natural Resources, Merritt Island, Florida, February 1979.

Capece, J.C., J.C. Campbell, and L.B. Baldwin. Estimating Peak Rates and Volumes from Flat, High Water Table Watersheds. Paper No. 84-2020, ASAE, St. Joseph, MI, June 1984.

Carlisle, V.W., M.E. Collins, F. Sodek, III, and L.C. Hammond. Characterization Data for Selected Florida Soils. Soil Science Research Report No. 85-1, Dept. of Soil Science, Univ. of Florida, Gainesville. 1985.

Carter, H.H. and Okubo, A. A Study of the Physical Processes of Movement and Dispersion in the Cape Kennedy Area. Report No. NYO-2973-1, Chesapeake Bay Institute Ref. 65-2. 1965.

Christensen, B.A. Hydraulics of Sheet Flow in Wetlands. Proc. 3rd Annual Sympos. of Waterways, Harbors, and Coastal Eng. Div., ASCE, RIVERS 76, Vol. 1, 746-756, Fort Collins, CO, 1976.

Christensen, B.A. Hydraulic Notes. Hydraulic Laboratory, Dept. of Civil Eng., Univ. of Florida, Gainesville, 1985.

Clark Engineers-Scientists, Inc. KSC Subsurface Hydrology and Groundwater Survey. The Site Specific Program Report, Miami, Florida, 1985.

Clark Engineers-Scientists, Inc. KSC Subsurface Hydrology and Groundwater Survey. Volume IV, The Areawide Program. NASA Pub. No. KSC-DF-3081. 1987.

Da Costa, S.L. and K.A. Glatzel. Simulating Nonpoint Source Runoff to Coastal Waters. ASCE, Proc. 5th Sympos. Coastal and Ocean Mgmt, Vol. IV, Seattle, Washington, 3903-3916, May 1987.

Delfino, J.J., D. Frazier, and J. Nepshinsky. Contaminants in Florida's Coastal Zone. Florida Sea Grant College Report No. 62. Gainesville, 1984.

Dendrou, S. and K.A. Cave. Accounting for Tidal Flooding in Developing Urban Stormwater Management Master Plans. Proc. of Stormwater and Water Quality Model Users Group Meeting, EPA/600/9-87/016, March 23-24, Denver, CO., 1987.

Diniz, E.V. and W.H. Espey, Jr. Maximum Utilization of Water Resources in a Planned Community. Application of the Storm Water Management Model, Volume I. EPA-600/2-79-050c, U.S. Environmental Protection Agency, Cincinnati, OH. July 1979.

Dreschel, T.W. B.C. Madsen, L.A. Maull, C.R. Hinkle, W.M. Knott. Precipitation Chemistry: Atmospheric Loadings to the Surface Waters of the Indian River Lagoon by Rainfall. In press, Florida Scientist.

Driscoll, E.D. In: Benefit Analysis for Combined Sewer Overflow Control. Seminar Pub. EPA-625/4-79-013, Environmental Protection Agency, Cincinnati, OH. April 1979.

Dunne, T. and L.B. Leopold. Water in Environmental Planning. W.H. Freeman and Co., San Francisco, CA. 1978.

Dwornik, D.S. A Stormwater Collection and Reuse System for the Kennedy Space Center. Florida Water Resources Research Center Pub. No. 83, University of Florida, Gainesville, 1984.

Ferrara, R.A. and P. Witkowski. Stormwater Quality Characteristics in Detention Basins. Proc. ASCE, J. Env. Eng. Div., Vol. 109, No. 2, 428-447, 1983.

Florida Department of Regulation. Biological Potential for Slowing, Stopping, and Reversing the Trend of Seagrass Loss. Seagrass Task Force "White Paper", Unpublished Report, 1987.

Florida Department of Transportation. Drainage Manual. Florida Department of Transportation, Tallahassee, Florida, 1987.

Fonseca, M.S., W.J. Kenworthy, and G.W. Thayer. Restoration and Management of Seagrass Systems: A Review. Proc. International Symposium on Ecology and Management of Wetlands, College of Charleston, Charleston, S.C., June 1986.

Goforth, G.F. Long-Term Performance of Stormwater Detention Facilities: A Comparison of Design Methodologies. Florida Water Resources Research Center Pub. No. 58, University of Florida, Gainesville, 1981.

Grizzard, T.J., C.W. Randall, B.L. Weand, and K.L. Ellis. Effectiveness of Extended Detention Ponds. Proc. Eng. Found. Conf. on Urban Runoff Quality, ASCE, New England College, Henniker, New Hampshire, 323-337, June 1986.

Heaney, J.P. Introduction. In: Introduction to Social Choice Theory for Environmental Decision Making. P.D. Straffin. ASCE Urban Water Resources Reserach Program Tech. Memo. No. 36, February 1979, New York.

Heaney, J.P. Cost-Effectiveness and Urban Storm-Water Quality Criteria. Proc. ASCE Conf. on Current Practice and Design Criteria for Urban Runoff Water Quality Control, New York. 1988.

Heaney, J.P., W.C. Huber, W.C. Downs, M.C. Hancock, and C.N. Hicks. Impacts of Development on the Water Resources of Cypress Creek, North of Tampa. Pub. No. 89, Florida Water Resources Research Center, Gainesville. 1986.

Heaney, J.P., W.C. Huber, R. Field, and R.H. Sullivan. Nationwide Cost of Controlling Combined Sewer Overflows and Urban Stormwater Discharges. Jour. Water Pollution Control Fed., Vol. 51, No. 8, Aug. 1979.

Heaney, J.P., W.C. Huber, and M.E. Lehman. Nationwide Assessment of Receiving Water Impacts from Urban Stormwater Pollution. Vols. I and II, EPA-600/52-81-025, Cincinnati, OH, 1981.

Heaney, J.P., B. Koopman, D.S. Dwornik, and R. Saliwinchik. Water Resources of the Kennedy Space Center. Florida Water Resources Research Center Pub. No. 82, Univ. of Florida, Gainesville. 1984.

Heffernan, J.J. and R.A. Gibson. A Comparison of Primary Production Rates in Indian River, Florida Seagrass Systems. Florida Scientist, Vol. 46, No. 3/4, 1983.

Holler, J.D. Storm Water Detention Basin Nutrient Removal Efficiency. Proc. ASCE, J. Water Resources Planning and Management, Vol. 115, No. 1, 52-63, 1989.

Huber, W.C. Deterministic Modeling of Urban Runoff Quality. In: Urban Runoff Pollution. H. Torno, J. Marsalek, and M. Desbordes, eds., NATO ASI Series, Series G: Ecological Sciences, Vol. 10, Springer-Verlag, New York, 167-242, 1985.

Huber, W.C. and R.E. Dickinson. Storm Water Management Model, Version 4. Office of Research and Development, U.S. Environmental Protection Agency, Athens, GA., 1988.

Huckle, H.F., H.D. Dollar, and R.F. Pendelton. Soil Survey of Brevard County, Florida. U.S. Department of Agriculture, Soil Conservation Service, Washington, D.C. 1974.

Hyde, H.C., R.S. Ross, and F. Demgen. Technology Assessment of Wetlands for Municipal Wastewater Treatment. USEPA, EPA-600/2-84-154, Office of Research and Development, Cincinnati, OH, 1984.

Hydroscience, Inc. A Statistical Method for Assessment of Urban Stormwater Loads-Impacts-Controls. U.S. Environmental Protection Agency, EPA 440/3-79-093, Washington, D.C., 1979.

James, W. and S. Boregowda. Continuous Mass-Balance of Pollutant Build-Up Processes. In: Urban Runoff Pollution. H. Torno, J. Marsalek, and M. Desbordes, eds., NATO ASI Series, Series G: Ecological Sciences, Vol. 10, Springer-Verlag, New York, 243-271, 1985.

Jewell, T.K., T.J. Nunno, and D.D. Adrian. Methodology for Calibrating Stormwater Models. Proc. ASCE, J. Env. Eng. Div., Vol. 104, No. EE3, 485-501, 1978.

Jones, K.L. Quantitative Assessment of Stormwater Runoff from the Vehicle Assembly Building Area of Kennedy Space Center, Florida. Master's Thesis, Florida Institute of Technology, Melbourne, 1986.

Laster, J.A. Water Chemistry Studies of the Indian River Lagoons. Report to NASA in An Ecological Study of the Lagoons Surrounding the John F. Kennedy Space Center, Florida. Florida Institute of Technology, Melbourne. 1975.

Levenspiel, O. Chemical Reaction Engineering. Second Edition. John Wiley and Sons, Inc., New York, New York, 1972.

Livingston, E.H. Stormwater Regulatory Program in Florida. Proc. Eng. Found. Conf. on Urban Runoff Quality, ASCE, New England College, Henniker, New Hampshire, 249-256, June 1986.

Manahan, S.E. Environmental Chemistry. Fourth Edition. Willard Grant Press, Boston, 1984.

Martin, E.H. Effectiveness of an Urban Runoff Detention Pond-Wetlands System. Proc. ASCE, J. Env. Eng. Div., Vol. 114, No. 4, 810-827, 1988.

McCuen, R.H. and G.E. Moglen. Multicriterion Stormwater Management Methods. Proc. ASCE, J. Water Resour. Plan. and Mgmt., Vol. 114, No. 4, 414-431, 1988.

Medina, M.A., W.C. Huber, and J.P. Heaney. Modeling Stormwater Storage/Treatment Transients: Theory. Proc. ASCE, J. Env. Eng. Div., Vol. 107, No. EE4, 1981a.

Medina, M.A., W.C. Huber, and J.P. Heaney. Modeling Stormwater Storage/Treatment Transients: Applications. Proc. ASCE, J. Env. Eng. Div., Vol. 107, No. EE4, 1981b.

Mion, L.P.O. Low-Energy Wastewater Recycling Through an Oligohaline Coastal Marsh Ecosystem. M.S. Thesis, Univ. of Florida, Gainesville, 1986.

Mote Marine Laboratory. Biological and Chemical Studies on the Impact of Stormwater Runoff Upon the Biological Community of the Hillsborough River, Tampa. Final Research Report submitted to Department of Public Works, Tampa, Florida. Mote Marine Laboratory, Sarasota, Florida. 1984.

National Aeronautics and Space Administration. Environmental Resources Document, John F. Kennedy Space Center, Florida, NASA Pub. No. KSC-DF-3080, 1986.

National Oceanic and Atmospheric Administration. Flood Insurance Study, Brevard County, Florida, 1971.

Nix, S.J. Analysis of Storage/Release Systems in Urban Stormwater Quality Management. Ph.D., Dissertation. Univ. of Florida, Gainesville, 1982.

Nix, S.J. Residence Time in Stormwater Detention Basins. Proc. ASCE, J. Env. Eng. Div., Vol. 111, No. 1, 95-100, 1985.

Nix, S.J. and J.P. Heaney. Optimization of Storm Water Storage-Release Strategies. Water Resour. Res., Vol. 24, No. 11, 1831-1838, 1988.

Nix, S.J., J.P. Heaney, and W.C. Huber. Suspended Solids Removal in Detention Basins. Proc. ASCE, J. Env. Eng. Div., Vol. 114. No. 6, 1331-1341, 1988.

Odum, W.E. and M.E. Hawley. Impacts of Urban Runoff on Estuarine Ecosytems. Proc. Eng. Found. Conf. on Urban Runoff Quality, ASCE, New England College, Henniker, New Hampshire, 182-191, June 1986.

Peffer, S.D. Sediments. Report to NASA in An Ecological Study of the Lagoons Surrounding the John F. Kennedy Space Center, Florida. Florida Institute of Technology, Melbourne. 1975.

Petryk, S. and G. Bosmajian. Analysis of Flow Through Vegetation, Proc. ASCE. J. Hydraulics Div., Vol. 101, No. HY7, 871-884, 1975.

Provancha, J.A. and R.M. Willard. Submerged Aquatic Vegetation Density Classes, John F. Kennedy Space Center, Florida. GIS Information, NASA/KSC Biomedical Operations and Research Office, 1986.

Provancha, M.J. P.A. Schmalzer, and C.R. Hinkle. Vegetation Types, John F. Kennedy Space Center. GIS Information, NASA/KSC Biomedical Operations and Research Office, 1986.

Rice, J.D., R.P. Trocine and G.N. Wells. Factors Influencing Seagrass Ecology in The Indian River Lagoon. Florida Scientist, Vol. 46, No. 3/4, 1983.

Roesner, L.A., J.A. Aldrich, and R.E. Dickinson. Storm Water Management Model User's Manual Version 4: EXTRAN Addendum. Office of Research and Development, U.S. Environmental Protection Agency, Athens, GA., 1989.

Ryan, J. and A. Goetzfried. Unpublished water quality data for the Buck Creek drainage network. NASA Environmental Management Staff, Design Engineering, John F. Kennedy Space Center, Florida, 1988.

Schilling, W. Urban Runoff Quality Management by Real-Time Control. In: Urban Runoff Pollution. H. Torno, J. Marsalek, and M. Desbordes, eds., NATO ASI Series, Series G: Ecological Sciences, Vol. 10, Springer-Verlag, New York, 765-817, 1985.

Smith, N. The Suitability of Routine Weather Data for Estimating Local Estuarine Heat Energy Fluxes. Estuaries, Vol. 8, No. 3, 270-278, 1985.

Smith, N, P.A. Pitts, and P.S. Mikkelsen. Indian River Lagoon Circulation and Flushing Study. Final Report to the Florida Department of Environmental Regulation, Contract No. CM-118, Harbor Branch Oceanographic Institution, Inc., 85pp, 1987.

Smolenyak, K.J. Urban Wet-Weather Pollutant Loadings. M.E. Thesis, Univ. of Florida, Gainesville, 1979.

Standard Methods for the Examination of Water and Wastewater, 16th ed., Amer. Publ. Health Assoc., Amer. Water Works Assoc., Water Pollut. Control Fed., Washington, D.C. 1985.

Stedinger, J.R. and M.R. Taylor Synthetic Streamflow Generation 2. Effect of Parameter Uncertainty. Water Resour. Res., Vol. 18, No. 4, 919-924, 1982.

Steward, J.S. and J.A. Van Arman, eds. Indian River Lagoon Joint Reconnaissance Report. St. Johns River Water Management District and South Florida Water Management District. Contract No. CM-137. 1987.

Stowell, R., S. Weber, G. Tchobanoglous, B.A. Wilson, and K.R. Townzen. Mosquito Considerations in the Design of Wetland Systems for the Treatment of Wastewater. In: Ecological Considerations in Wetlands Treatment of Municipal Wastewaters, P.J. Godfrey, E.R. Kaynor, S. Pelczarski, and J. Benforado, eds. Van Nostrand Reinhold Company, New York, 1985.

Straffin, P.D. Introduction to Social Choice Theory for Environmental Decision Making. ASCE Urban Water Resources Research Program Tech. Memo. No. 36, February 1979, New York.

Suphunvorranop, T. and D.A. Clapp. A Preliminary Study of Pollutant Concentrations for Turkey Creek Basin. St. Johns River Water Management District Tech. Pub. SJ 84-11, Palatka, FL. 1984.

Thompson, M.J. Species Composition and Distribution of Seagrass Beds in the Indian River Lagoon, Florida. Florida Scientist, Vol. 4, No. 2, 1978.

Trefry, J.H., R.P. Trocine, S. Metz, N. Iricanin, and N-C. Chen. Sediment and Contaminant Sources to the Indian River Lagoon: The Turkey Creek Model. The Second Indian River Research Sympos., Florida Institute of Technology, Melbourne, FL, Sept. 1988.

Trefry, J.H., D.K. Stauble. Origin, Composition and Fate of Organic-Rich Sediments in Coastal Estuaries, Project Muck (R/IRL-2). Final Report to The Florida Sea Grant College and The State of Florida Department of Environmental Regulation. Florida Institute of Technology, Melbourne, 1987.

Turner, A.K., K.J. Langford, M. Win, and T.R. Clift. Discharge-Depth Equation for Shallow Flow. Proc. ASCE, J. Irri. and Drain. Div., Vol. 104, No. IR1, 95-110, 1978.

U.S. Army Corps of Engineers, Evaluation of Oceanographic, Hydrographic, and Hydrologic Effects-Cape Canaveral Area. Florida, Jacksonville District, June 1962.

U.S. Army Corps of Engineers. Storage, Treatment, Overflow, Runoff Model, STORM User's Manual. Hydrologic Engineering Center, Davis CA, August 1977.

U.S. Environmental Protection Agency. Final Report of the Nationwide Urban Runoff Program. Water Planning Div., Washington, D.C. 1983.

U.S. Environmental Protectiion Agency. Design Manual, Constructed Wetlands and Aquatic Plant Systems for Municipal Wastewater Treatment. EPA/626/1-88/022, Office of Research and Development, Washington, D.C., September 1988.

U.S. Geological Survey. 1:24,000 quadrangle map for Orsino, Florida, Department of Interior, Washington, D.C., 1976.

U.S. Geological Survey. National Handbook of Recommended Methods for Water-Data Acquisition. USGS Office of Water Data Coordination, Reston, VA, 1980.

Viessman, W. and M.J. Hammer. Water Supply and Pollution Control. Fourth Ed. Harper & Row Publishers, New York, 1985.

Viessman, W., J.W. Knapp, G.L. Lewis, and T.E. Harbaugh. Introduction to Hydrology. 2nd Edition. Harper & Row Publisher, New York, 1977.

Whipple, W. and J.V. Hunter. Settleability of Urban Runoff Pollution. J. Water Pollu. Control Feder., Vol. 53, No. 12, 1726-1731, 1981.

Yousef, Y.A., T. Hvitved-Jacobsen, M.P. Wanielista, and R.D. Tolbert. Nutrient Transformation in Retention/ Detention Ponds Receiving Highway Runoff. Jour. Water Pollu. Control Feder., Vol. 58, No. 8, 838-844, 1986.

BIOGRAPHICAL SKETCH

Drew Burton Bennett was born on September 26, 1960, in East Hampton, Long Island, New York, to Mr. and Mrs. G. Berkley Bennett. He has one brother, G. Berkley, Jr. He was raised on Apaquogue Road in East Hampton where his favorite recreational activities were surfing, duck shooting, and surfcasting. In 1974 he received the East Hampton Ladies Village Improvement Society Merritt Award for interest in the environment. He graduated from East Hampton High School in 1978. It was the "island living" that drew his interest to coastal water resources. In 1982, he graduated from The University of New Hampshire with a Bachelor of Science degree in hydrology. In 1984 he moved to Florida where he began his professional career with an environmental engineering firm at NASA's John F. Kennedy Space Center. There he gained great interest in surface-subsurface water interactions and its implication in water resources engineering and management. Pursuing this, he enrolled in the Graduate School at The University of Florida in 1987, earning a Master of Science degree in environmental engineering sciences with a concentration in water resources engineering in 1989. His plans after graduation include buying a new surfboard and moving to Boston, Massachusetts, to continue his professional career.