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COMPUTER-AIDED ENGINEERING APPROACH TO AGRICULTURAL FLOOD HAZARD MANAGEMENT IN FLORIDA

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L. B. Baldwin, K. L. Campbell, A. B. Bottcher, John Capece,
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Final Technical Completion Report

COMPUTER-AIDED ENGINEERING APPROACH TO AGRICULTURAL FLOOD HAZARD MANAGEMENT IN FLORIDA

March 1987

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INTRODUCTION

Stormwater management has been a necessary part of development in most of South Florida, due to the low, flat terrain and characteristic high-water-table soils. For nearly a century, surface water control could be achieved through application of well understood engineering technology with little concern for any alteration of the natural ecosystems. In recent years, however, it became evident that land development, urban and agricultural, was impacting off-site water bodies as well as consuming on-site habitats of wildlife and native vegetation. Wave after wave of regulations initiated by the state legislature have been directed at saving some of the natural features of the region, such as wetlands, and protecting surface waters from degradation due to pollutants in stormwater runoff. These regulations have imposed several constraints on stormwater management but the control of water remains a necessity for most uses of land which support the growing human population.

Increasing stormwater runoff and wetlands protection regulations which control agricultural development activities in the high-water-table soils of south-central Florida, have created a need to assess our understanding of the hydrology of the area and the applicability of conventional design consideration to the design of water management systems. Developers striving to meet regulatory as well as economic requirements are not currently given clear objective functions to guide their system design. One example is found in Appendix 7, Isolated Wetlands, of "Basis of Review for Surface Water Management Permit Application", a guideline publication of the South Florida Water Management District (SFWMD) (1987). Language refering to "...isolated wetlands and their associated fish and wildlife functions and values," provides little criteria for identifying or quantifying these functions and provides even less insight as to how specific water management systems will affect these functions. This is primarily an admission that the isolated wetlands and the watersheds which contain them are sensitive, but the degree of that sensitivity is not fully understood. Therefore particular caution must be exercised in developing and managing these areas and the burden of proof of due caution is imposed on developers and their engineers.

Reasonable caution has dictated requirements in the isolated wetlands guideline referenced above such as:

5.1.6 a. Water tables shall not be altered such that on-site and off-site isolated wetlands are adversely affected.

b. Minimum separation distance between wetlands and canal/lake excavations shall be 200 feet, unless soil or other data shows that water table elevations in the wetlands would not be adversely affected.

c. Control elevations shall be established which maintain or improve pre-development hydroperiods in protected wetlands.

5.1.7 a. Buffer zones may be required around all isolated wetlands that are to be protected or incorporated into a surface water management system to protect wetland function and minimize adverse impacts of upland development on wetland function. Actual delineation of the buffer zone may vary according to site specific conditions. Buffer zones which extend at least fifteen feet landward from the edge of the wetland in all places and average twenty-five feet from the landward edge of the wetland will be presumed adequate.

Another provision in the guideline sets one-half acre as the size threshold for habitat impact review. Isolated wetlands smaller than one-half acre may be ignored up to only 3% of proposed development area. Wetlands between one-half and five acres are to be protected or mitigated by development of replacement habitat elsewhere. Isolated wetlands over five acres in size are considered less likely candidates for mitigation, and are to be protected. It is not unusual for flatwoods in South Florida to contain wetlands over 15-20% of large tracts.

It is evident that wetland preservation and water quality as well as quantity criteria must be considered in any planning for high-water-table land development and associated water management systems. Regulations lead engineers and planners to contemplate questions such as: What constitutes adverse water table effects? How might one successfully gather and analyze soils and other data to quantitatively describe effects of a given water management system? Without historical data, how does one know whether hydroperiods are being modified? What plant community alterations will result from development and are these implications now unacceptable or could they be in the future judged unacceptable? How do I merge these environmental and agricultural/economic objectives? These are but a few implications of the myriad of rules governing agricultural development involving surface water management. Very quickly the designer realizes that all applicable constraints must be assembled and a true systems approach employed to arrive at a successful development strategy.

The planner should begin by assessing constraints pertaining to the natural layout and physical characteristics of the property to be developed. Which portions of the property carry with them stringent regulatory criteria? What are the associated limitations regarding water quality, stormwater runoff, hydroperiods, water tables, etc.? With this appreciation, information and data can then be intelligently assembled to provide an adequate assessment of pre-development conditions.

Next the engineer/planner can begin to decide how to merge the constraints and objectives. Can existing wetlands be integrated into the overall water management system without threatening their functional value? A methodology, acceptable to the regulatory authorities, for assessing the operating characteristics and implications of any proposed water management systems must be identified or established. Given this, the pieces of the puzzle: production areas, ditches, control structures, artificial impoundments, and natural wetlands, can be assembled to arrive at system alternatives. Evaluation of these scenarios may dictate that mitigation schemes are required. Again stated, the ability of the engineer/planner to successfully complete the design process hinges on his or her understanding of the system and the availability of tools to facilitate application of that knowledge.

PURPOSE OF PROJECT

Although many of aspects of stormwater management system design mentioned above are new, considerable research has been carried out in typical high-water-table soil areas of South Florida which will help engineers and planners with design. The purpose of this study is to address some of the design considerations involved in developing large tracts of flatwood soils for intensive agriculture with economical protection against flooding and crop damage. Citrus grove development is currently active in the area as a result of massive freeze damage in plantings farther north, and the general economic pressure to make land more productive. While not exclusively for citrus grove design, this project proceeded with that level of stormwater control in mind.

The project goals are to develop improved methods for defining:

1) Effects of soil-water storage and evaportranspiration from cropland and wetland on stormwater runoff.

2) Integration of major storm runoff with stormwater detention/retention requirements.

3) Nutrient attenuation in water management systems.

Items 2) and 3) involved development of computer models which are compatible with some previous modeling of water quantity and quality for high-water-table soils. Future work is planned to link several models in a comprehensive design package addressing factors covered in this study, and other design considerations.

The three items listed above are treated in Parts I, II and III which follows.

PART I EFFECTS OF SOIL-WATER STORAGE AND EVAPOTRANSPIRATION FROM CROPLAND AND WETLAND ON STORMWATER RUNOFF

Hydrologic Regime

Runoff from the flat, high-water-table areas of South Florida is highly dependent upon the soil-moisture status reflected by the surficial water table level. This surficial water table is dynamic, but seldom drops below 6 feet from the surface. The temporal fluctuations are known to be a function of soil-moisture sources and sinks, namely: rainfall, surface runoff, deep seepage, lateral subsurface flow, and evapotranspiration (ET). Of these, deep seepage is commonly considered to be negligible in high-water-table soils. At distances of more than one or two-hundred feet, head boundary conditions (ditches, ponds, etc.) are not believed to induce significant subsurface lateral flow. This leaves runoff and ET as the primary sinks for the system as we currently understand it. The following discussion will focus on the various hydrologic component interactions and mechanisms, and in particular the effects of soil-moisture storage and ET from crops and wetlands on runoff and associated design considerations.

Soil Classification:

Approximately one-third of Florida is classified as having flatwoods-type soils, which are of the spodosol order. Spodosols are mineral soils which have a spodic horizon, a subsurfance horizon with an accumulation of organic matter and oxides of aluminum with or without iron oxides. The presence of a spodic horizon indicates impermeable subsoil and little or no deep seepage. Aqouds is the specific suborder of Spodosols occurring in Florida. These occur in areas which are seasonally saturated with water with slopes ranging from 0 to 2 percent but are typically less than 0.5 percent. Where drained, these areas can support citrus and other special crops (Brady, 1974). Flatwoods soils typically have very high hydraulic conductivities (>6 in/hr). Internal drainage may be very rapid to slow, depending on the amount of ditching present. Therefore, a hydrologic classification of A/D or B/D is used, with the appropriate classification determined by the effectiveness of drainage improvements at lowering the water table, which varies with topography and soil classification and generally ranges from the surface to 6 feet deep.

Three general geographic classifications of flatwoods occur in Florida: the Gulf Coast Flatwoods, Atlantic Coast Flatwoods, and the Southern Florida Flatwoods as shown in Figure I-1. Wetlands are an important component of these areas, impacting both hydrology and water quality. There are many isolated ponded areas, some seasonal and many year round. Due to the extremely low watershed slopes, these areas often remain wet even when ditched. Another effect of the low watershed slopes is to increase the difficulty in delineating watershed boundaries. Drainage patterns can, in fact, shift depending upon rainfall patterns and runoff magnitude.

Hydrologic Processes:

In classical hydrology, three concepts, overland flow, interflow, and base flow, are used to describe the routes which water travels from the time it reaches the ground until it enters a stream channel. These concepts are difficult to apply to flatwoods watersheds. As rainfall enters the ground, a rapid rise in the water table occurs as a result of the high infiltration capacity of the sandy soils and the presence of impermeable subsoil which greatly limits deep seepage. As the water table rises, subsurface flow increases if ditches are present to induce a gradient. This has been described as interflow as opposed to base flow which connotes a longer time period. Runoff primarily results from rainfall on saturated areas which occur as the result of the presence of the water table at the surface and on the many wetland areas which are present on the watershed; and from subsurface flow to ditches and drainage ways both during and following a storm. More appropriate terms for describing runoff from flatwoods watersheds are rapid, intermediate, and slow flow. These terms do not attempt to describe processes but refer only to flow rates (Speir et al., 1969).

Runoff from these areas is not produced by the common concept of an infiltration limiting "rainfall excess." Flatwoods watersheds respond primarily as a "storage-based" system (Heatwole, 1986). Although the watersheds are storage based, it is not a strict storage relationship. Lower areas of a watershed (such as sloughs) begin contributing runoff before the entire watershed is saturated. This is similar to the "variable source area" concept (Hewlett and Nutter, 1970; Hewlett and Troendle, 1975; Betson and Marius, 1969). These researchers were studying forest hydrology where infiltration rates were also rarely, if ever, a limiting factor.

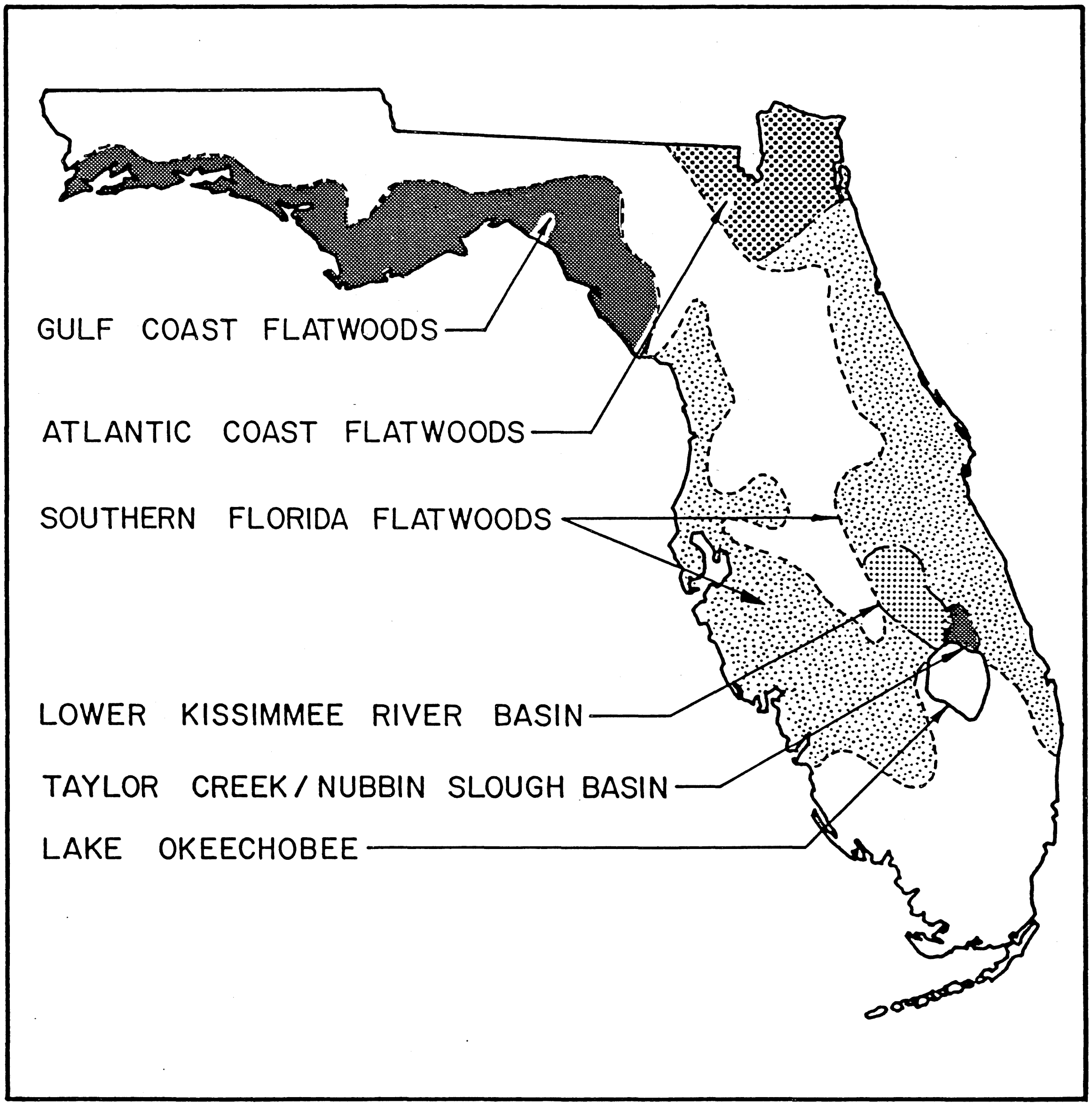

Figure I-1. General classification and distribution of flatwoods soils in Florida (Brady, 1974) and study area location.

Water Table - Runoff Relations

Capece (1984) evaluated several implementations of the SCS runoff equation and arrived at a generalized relationship between depth to the water table, 24-hour rainfall depth and runoff volume (Figure I-2). Built into this graph is a relationship between depth to the water table and available soil moisture storage as developed by the USDA-ARS (Speir et al., 1969), shown in Figure I-3 and interpreted in Figure I-4. Other water table-available soil moisture relations have been developed, notably by the SFWMD (Figure I-5). The District curve differentiates between natural-undisturbed and developed-compacted soils while the ARS curve corresponds to natural-undisturbed conditions. Of the two, runoff volume estimates based upon the ARS relationship more closely agreed with observations from watersheds of the Upland Detention Demonstration Project.

The SFWMD and ARS avaialable storage curves describe a static situation and do not quantify the dynamics of the subsurface moisture resevoir ie., rise and fall of the water table; nor does Figure I-2 give an indication as to how the soil moisture status affects runoff rates.

The relationship between the quantity of water stored in the shallow surficial aquifer and the water table level is soil specific and a function of porosity of the various soil horizons, the areal and vertical distribution of these horizons, the water table level and the capillary fringe height. While calculation of soil moisture is relatively simple given these data, rarely is such information available.

In estimating changes in the water table resulting from extraction of a known volume of water, a constant drainable porosity parameter is typically, but incorrectly, employed. Overman and Zakariah (1974) show a method based on soil moisture characteristic curves which properly calculates changes in water table level given moisture extraction volume or vise-versa. Such an approach will yield a desorption curve similar to that shown in Figure I-3.

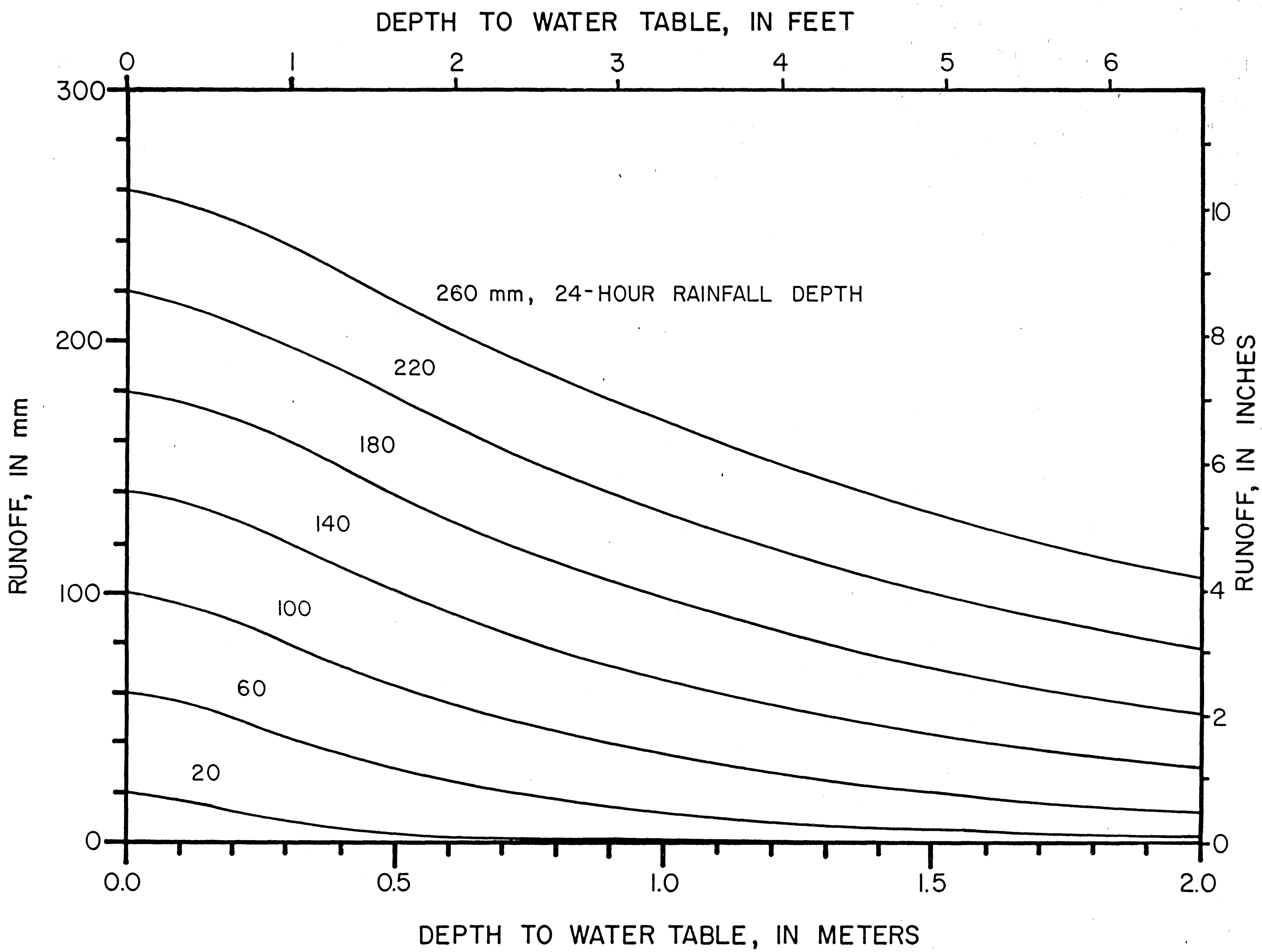

Figure I-2. Solution of the SCS runoff equation using watershed storage parameters as determined from Figure I-3 (ARS method).

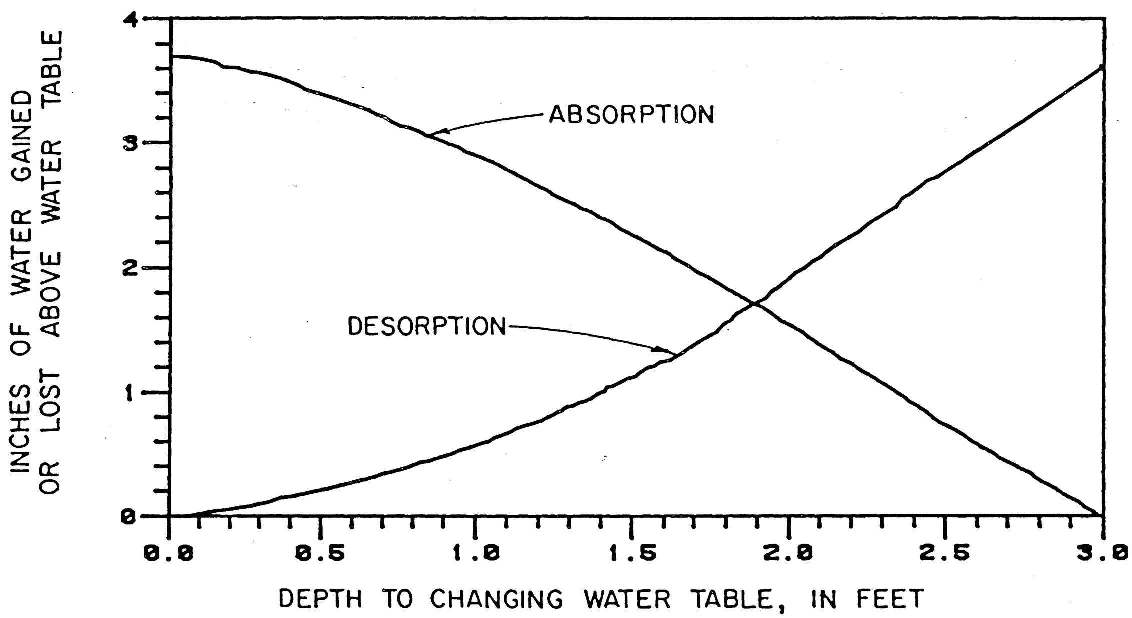

Figure I-3. Absorption/desorption characteristics for sandy soils of the Taylor Creek area (Speir et al., 1969).

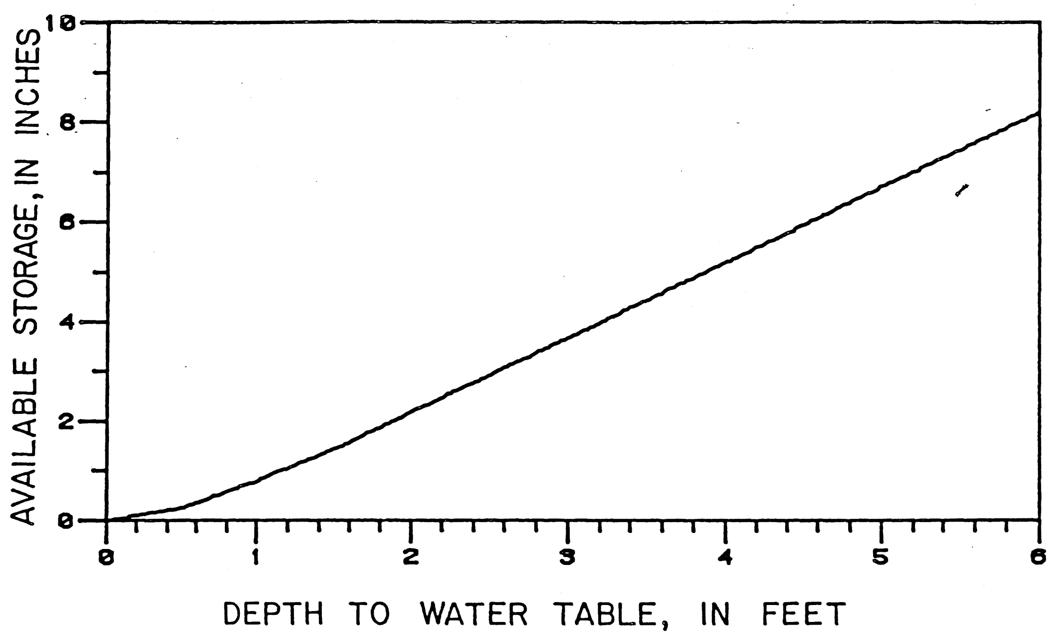

Figure I-4. Soil profile moisture storage capacity as a function of depth to the water table for use in the ARS method.

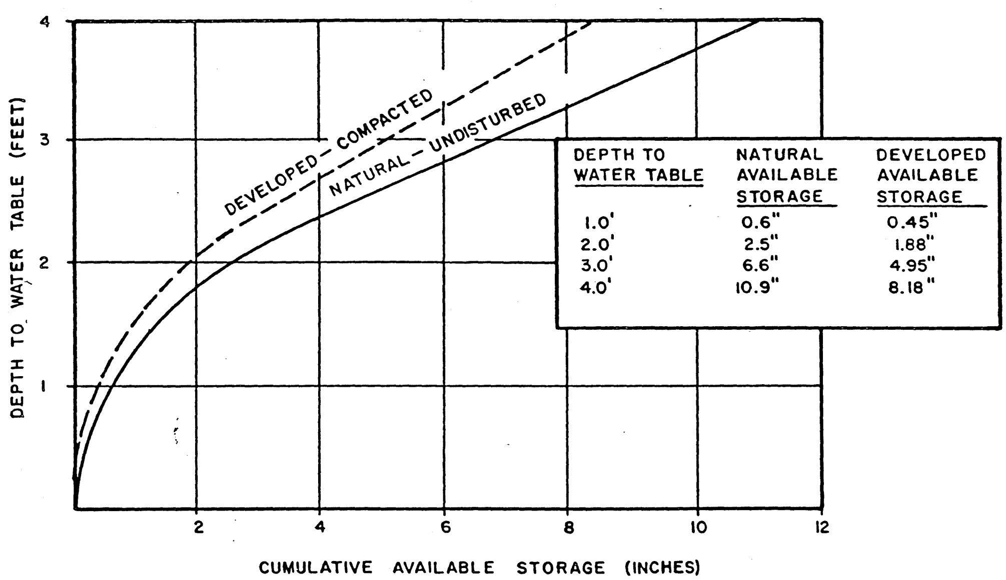

Figure I-5. Available soil profile storage for natural and developed watersheds of South Florida (SFWMD, 1983).

The inverse calculation, determining water table rise given a precipitation volume, is more complex, however Speir et al. (1969) analyzed field data and published the simplified formula:

$$Y = 0.87X - 0.26$$

where Y = water table rise in feet,

and X = precipitation depth in inches.

Investigations were also conducted by Capece (1984) to determine whether incorporating soil moisture status into the SCS Unit Hydrograph method would result in improved runoff hydrographs. Results did not reflect any improved predictive ability indicating that the primary mechanism by which available soil moisture storage affects runoff rate is via runoff volume. Despite this current lack of evidence, analysis of runoff mechanisms can lead to some hypotheses.

Stormwater runoff contributory factors include precipitation (volume and areal/time distribution), surface storage conditions, vegetative factors (as they affect interception, surface detention and roughness), watershed drainage density, slope, and geometry. Surficial aquifer levels can potentially influence runoff rates by affecting surface storage, particularly in low-lying wetlands. Water table levels can also influence vegetative interception, detention and roughness by encouraging or discouraging plant growth via moisture availability in the root zone.

Wetlands-Watertable Relations

Capece (1984) studied water table hydrographs measured over a three-year period on six sites and attempted to evaluate a model to simulate these hydrographs. Two components form this model: recession and rise. Recession of the water level is based on the curve published by Speir et al. (1969). Shown in Figure I-6, while water level rise is based upon the absorption curve shown in Figure I-3. The model is generalized for watersheds of the lower Kissimmee River Basin, considering only precipitation and time. It does not attempt to introduce specific soil type, vegetation, wetlands or other watershed and weather factors.

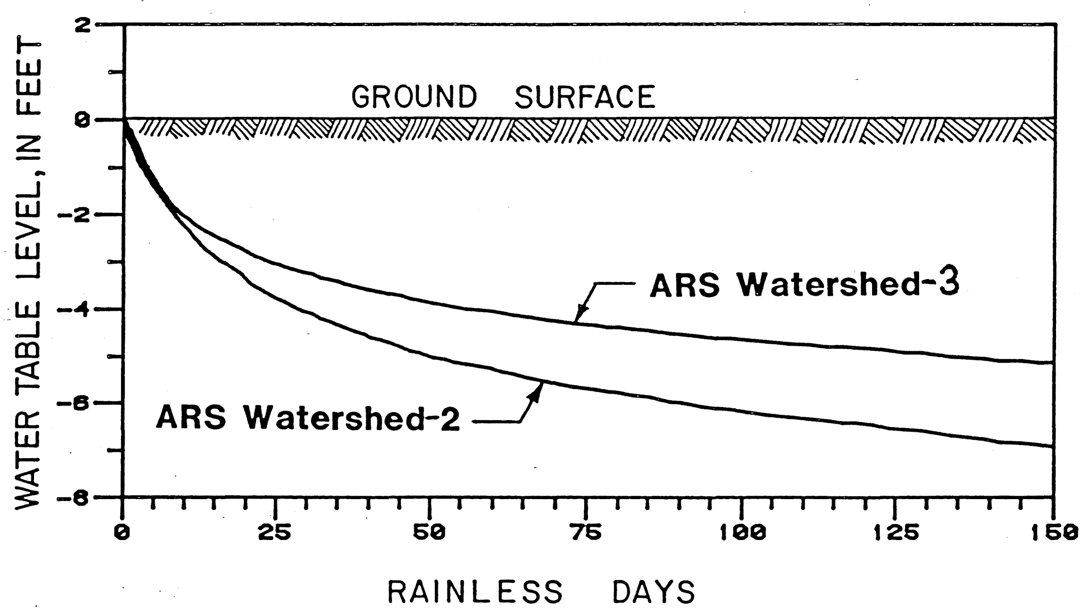

Figure I-6. Water table recession curves (Speir et al., 1969).

Results hinted that the model as currently implemented performed better during wet to average years than during dry years for watersheds with low percent wetlands. Where percent of the watershed covered by wetlands was high (>20%), the model performed better during dry years.

Further refinement of these water table models is required before they can be considered reliable for modeling impacts of agricultural development.

Wetlands-Runoff Relations

Wetlands play a significant storage role in the stormwater runoff process. Evidence of this appeared in studying the hydrology of the Upland Detention Demonstration Project watersheds. Capece (1984) found that by introducing a parameter reflecting watershed percent wetlands into the SCS Unit Hydrograph Method, observed hydrographs could be better modeled. The particular avenue for incorporation of this parameter was the "watershed lag" equation which leads to the "time- to-peak" unit hydrograph property. While the relationship between runoff hydrographs and wetlands is by no means refined, a tool for modeling the interaction does exist.

Evapotranspiration-Water Table Relations

Given techniques for estimating runoff from known water table conditions and for estimating changes in water table levels from known moisture extraction, one is led to the task of estimating the rate of water extraction from the soil profile through evapotranspiration (ET). Evapotranspiration is a two-tiered process. First, environmental conditions (temperature, solar radiation, relative humidity, etc.) establish a ceiling on the magnitude of ET, referred to as potential ET or PET. For the south-central Florida area, average PET values are shown in Figure I-7. Applicable PET estimation methods for Florida are presented by Shih et al. (1983) and Clark and Smajstrla (1983). The degree to which PET is limited (the secondary process) is often lumped into a coefficient assigned a value less than 1. Contributing to this coefficient are limitations attributable to vegetative conditions (species, root depth, and canopy coverage among others), and moisture availability (depth to the water table and soil type).

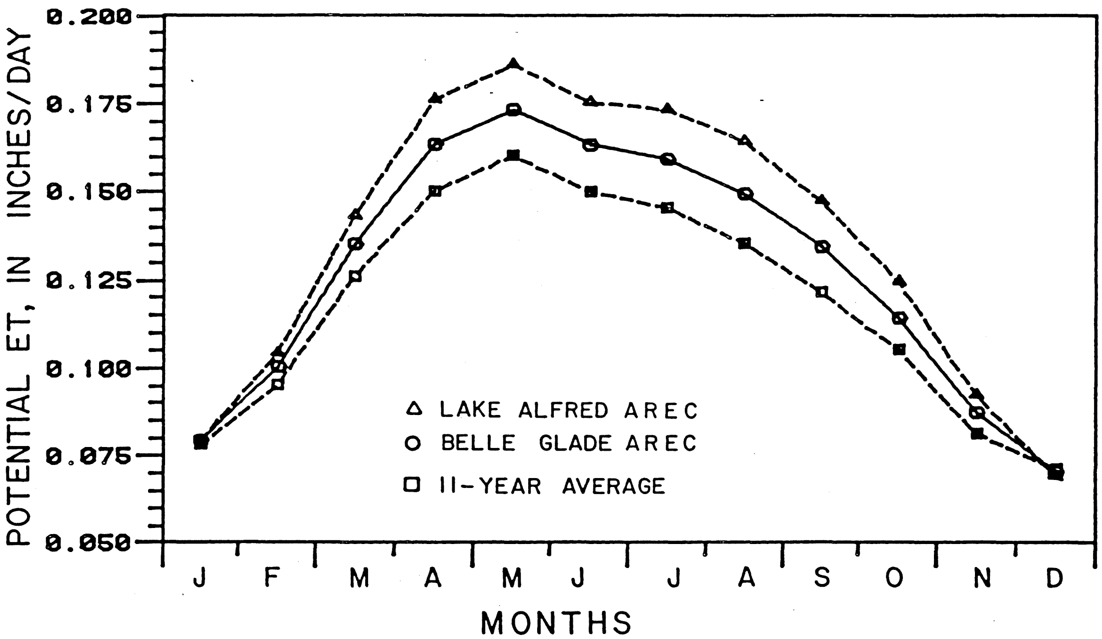

Figure I-7. Potential ET rates for the study area based upon an 11-year average of data from two locations (USDC-NOAA, 1972-82).

Despite ample rainfall, moisture availability is often limiting in sandy soils. Due to the conductivity properties of sandy soils, moisture is not readily transmitted to the soil surface when the water table and capillary fringe drops much below the soil surface. A method for determining water transport from a water table in sandy soils is presented by Overman and Zakariah (1974). Again, while methods are available, few documented applications or data sets to facilitate application are available for regional soils.

Crop-Evapotranspiration Relations

The CREAMS-WT hydrologic model (Heatwole 1986) considers the effects of vegetation in its determination of ET. This is accomplished by first estimating PET with a modified Penman equation. Leaf-area-index (LAI), radiation, and available soil moisture determine actual ET from potential ET. A simple plant root growth model is used to partition the ET extractions of soil water among the different soil layers. The CREAMS-WT ET algorithm is very sensitive to the LAI, radiation and temperature input data. For natural or improved pasture conditions, LAI changes little. However for other agricultural operations, LAI will vary significantly and becomes the most important parameter to be estimated (Heatwole, 1986).

Summary

The tools described above exist but all data and parameters necessary for their application do not, nor are these tools as yet assembled into a concise methodology. They therefore constitute design considerations as opposed to constituting a comprehensive design procedure. Work continues toward identification of methods, adaptions for regional use, assemblage of necessary data, and formulation of well-defined procedures. Specific contributions to these goals have been made as part of this project and are described in the following parts.

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II INTEGRATION OF MAJOR STORM RUNOFF WITH DETENTION/RETENTION REQUIREMENTS

Introduction

Stormwater management is defined as the process of controlling or manipulating storm runoff from the land surface through a combination of structural and nonstructural drainage and flood mitigation measures (Debo, 1980). Stormwater management was the earliest water management problem in Florida. Recent emphasis on water quality has resulted in stormwater management criteria requiring detention/retention of moderate storm runoff. There is a growing concern that current design guidelines may not provide adequate flood hazard protection, and a perceived need to develop designs that provide both quantity and quality control. The water management districts, the Department of Environmental Regulation, and local and county agencies are very concerned about this problem because they are trying to improve procedures for permitting the many applicants for new developments in Florida.

Urban development, recent losses of citrus by freezes, and economic pressures are forcing intensification of agriculture on marginal and poorly drained lands in South Florida. Stormwater control is essential to successful crop production on high-water-table soils. Discharge water quality constraints and the need to maintain natural hydroperiods have forced stormwater management beyond straight-forward hydrology and hydraulics, prompting the need for design guidelines which integrate environmental and water resource management requirements with conventional engineering solutions to stormwater control. In addition, the cost of stormwater management systems in terms of planning, area lost to production, construction, and operation and maintenance make it essential for the designer to evaluate several alternatives in selecting the most cost-effective system. These factors have prompted the need to bring together the latest findings of hydrology and computer modeling methods which are applicable to Florida's high-water-table soils and to develop stormwater management design procedures which incorporate the most up-to-date information available.

Objectives

The purpose of this study was to integrate research findings on hydrology and computer modeling methods most applicable to Florida's flat, high-water-table soils and to offer improvements to stormwater quantity management design procedures which incorporate the most complete information available. With these goals in mind, the following objectives are outlined:

A) Identify the best available procedures for modeling the hydrology of flat, high-water-table watersheds (Capece, 1984; Heatwole, 1986)

B) Develop computer models capable of analyzing various stormwater management system configurations for agriculture

Modeling Stormwater Runoff from Florida's Flatwoods

Storm Runoff Volume:

In the majority of the country, the concept of limited infiltration producing runoff is a valid one. However, flatwoods watersheds respond as a storage-based system (Heatwole, 1986). The most common storage-based method of predicting runoff volumes is the curve number method which was developed by the Soil Conservation Service (SCS). The curve number method was found to be a good model for flatwoods watersheds (Heatwole, 1986).

The SCS curve number method of estimating runoff was developed to provide an easy way to compute runoff while accounting for differences due to soils, land use, and management practices. Due to its simplicity, it is widely used.

The SCS runoff equation is written as:

$$Q = \frac{(P - 0.2S)^2}{(P + 0.8S)} \qquad [II\text{-}1]$$

where

Q = runoff

P = rainfall

S = watershed storage parameter.

The parameter S is usually determined by way of a "curve number" which varies between 0 and 100 as a function of several watershed factors namely 1) predominant soil types, 2) the soil's infiltration properties, 3) vegetative cover, 4) antecedent moisture conditions of the soil, and 5) land use and management practices. The curve number, CN, is related to S by the relationship:

$$CN = \frac{1000}{S + 10} \qquad [II\text{-}2]$$

Guidelines for the determination of the runoff curve number are given in Appendix II-1. A history of the origin and development of the SCS runoff equation has been presented (Rallison, 1980).

Capece (1984) evaluated several methods of predicting storm runoff volume to determine their applicability to flatwoods watersheds. His analysis was based on data from the Upland Detention/Retention Demonstration Project (Goldstein, 1986). The methods analyzed were all forms of the SCS runoff equation (Equation II-1) which differed in their approach to determining the storage parameter S.

The most accurate methods tested related S to the actual physical storage available in the soil profile. Curves were developed which related actual physical storage available as a function of water table depth. The "ARS" curve (Figure II-1) was developed from data observed in the Taylor Creek watershed (Speir et al., 1969). The South Florida Water Management District (SFWMD, 1983) developed curves for storage on natural sites and for developed sites (Figure II-2). The ARS curve predicts less available storage than does SFWMD curve, especially at water table depths greater than 2 feet. Capece (1984) found that using the ARS curve to estimate available storage gave better estimates of runoff volume.

Heatwole (1986) developed CREAMS-WT in order to reflect the hydrologic characteristics typical of flat, sandy, high-water-table watersheds such as those found in South Florida's flatwoods. The CREAMS-WT, a modified version of the water quality model, CREAMS (Knisel, 1980), uses a modified curve number method based on the ARS curve for predicting runoff.

The CREAMS-WT runoff equation is written as:

$$\frac{Q}{P} = \frac{[1 - 0.2(S_{MX}/ULE)*(S_{AV}/P)]^2}{[1 + 0.8(S_{MX}/ULE)*(S_{AV}/P)]} \qquad [II\text{-}3]$$

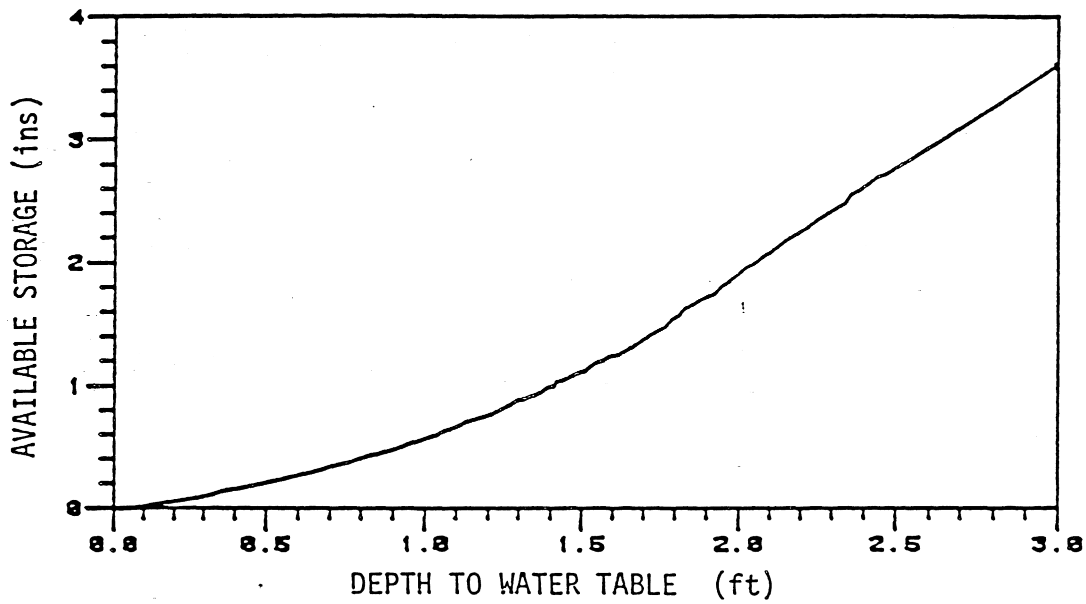


Figure II-1. Water storage characteristics of sandy soils from the Upper Taylor Creek watershed (Speir et al., 1969).

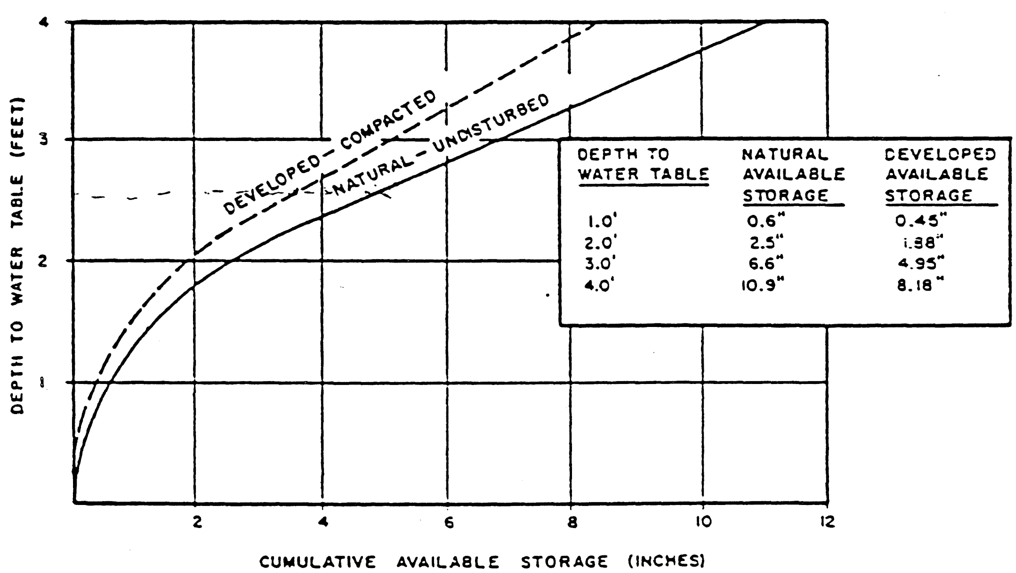


Figure II-2. Available soil profile storage as a function of depth to the water table for watersheds in South Florida (SFWMD, 1983).

where

S_{MX} = $1000/CN_I - 10$ = maximum value of S

CN_I = curve number for antecedent moisture condition I

ULE = upper limit of soil water storage in the profile

S_{AV} = available storage in the profile based on ARS curve.

The curve number (condition II) is an input parameter and is converted in the model to the equivalent CN_I value. The relationship between CN_{II} and S_{MX} is described in Appendix II-2.

Parameters for CREAMS-WT are physically based and can be estimated from various sources: the CREAMS manual, Soil Survey reports, Heatwole (1986), Capece (1984), and CREAMS-WT Users Manual (Heatwole, 1986). The CREAMS-WT runoff algorithm has the additional variable, the S_{MX}/ULE ratio. The effect of this variable is shown in Figure II-3.

Heatwole (1986) found the curve number method as used in CREAMS-WT to be appropriate for predicting runoff from flat, high-water-table watersheds. It represents the variable source area concept which has a significant impact on runoff from flatwoods watersheds and describes the rainfall/runoff process on flatwoods watersheds. The CREAMS-WT can also simulate the dynamic water table, limit deep seepage, and compute a total water balance on a continuous basis.

Storm Runoff Peak Rate:

An accurate estimate of the peak runoff rate is important in stormwater management in that regulatory criteria generally requires that the peak runoff rate from the developed site not exceed that which occurred before development. A variety of approaches are available for arriving at the peak runoff rate.

Capece (1984) examined the peak runoff rates from several small agricultural watersheds in the Lower Kissimmee River basin and the Taylor Creek-Nubbin Slough basin. These watersheds were typical of flatwoods watershed in general. Capece (1984) found that discharge hydrographs from

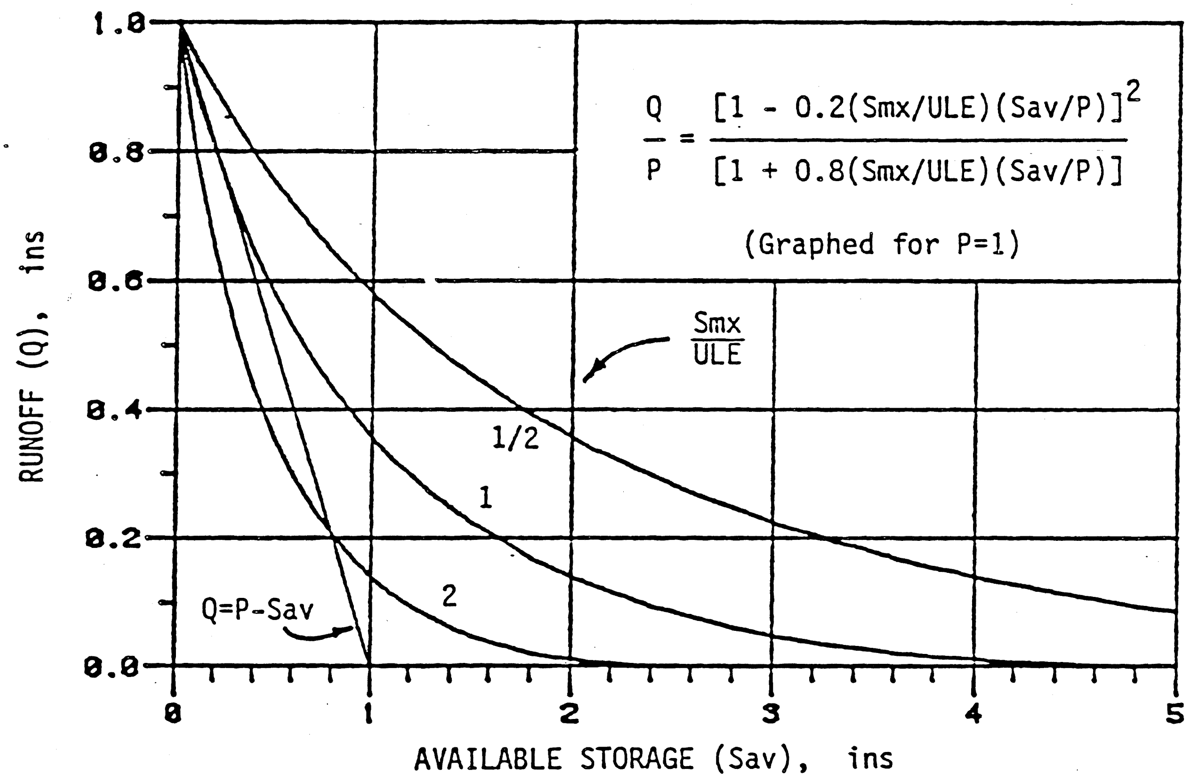


Figure II-3. Runoff predicted by: the CREAMS-WT runoff equation (graphed for several values of Smx/ULE), and the linear, strict storage relationship of Q=P-Sav. These functions are graphed considering unit precipitation (P=1).

flatwoods watersheds are much more attenuated and produce much lower peaks than most other small watersheds of the United States. This is due primarily to the extremely flat slopes (<0.5 percent) and the large amount of wetland storage present on these watersheds.

Capece (1984) evaluated the performance several methods of determining peak runoff on flatwoods watersheds and found that, in general, the more physically based models gave better results. However, when empirical models were tailored to specific watershed conditions, results were comparable to those from more complex models. Best results were obtained using the SFWMD overland flow computer program, a modified peak rate equation taken from CREAMS, and a modified SCS unit hydrograph method.

SFWMD model. The SFWMD overland model was constructed by Higgins (1976) and implemented by SFWMD (1979). The model uses Manning's form of the overland flow momentum equation combined with an assumed retention depth:

$$q = W(\frac{1.49}{n})(D - D_r)^{1.67} S^{0.5} \qquad [II-4]$$

where

q = watershed outflow in cfs

W = watershed width in ft

n = Manning's roughness coefficient

D = surface water depth in ft

S = watershed ground slope in ft/ft

D_r = watershed retention depth in ft.

The watershed is modeled as a single uniform inclined plane with continuity calculated using the following scheme:

$$D_i = D_t + R\Delta t - f\Delta t \qquad [II-5]$$

$$D_{t+1} = D_i - (\frac{(q(D_i)\ \Delta t\ 3600)}{A}) \qquad [II-6]$$

where

D_i = intermediary water depth in ft

D_t = initial water depth in ft

Wt = simulation time increment in hours

R = rainfall rate in ft/hr

f = infiltration rate in ft/hr

D_{t+1} = final water depth in ft

$q(D_i)$ = outflow rate calculated at D_i in cfs

A = watershed area in ft^2.

Watershed outflow rate calculation begins when D_i exceeds D_r and continues for each time increment until D_i again approaches D_r. Rainfall is assumed to follow the SFWMD distribution (Figure II-4). Infiltration is calculated using Horton's equation with an initial rate of 3.1 in/hr and a final rate of 0.01 in/hr. In Horton's method, infiltration rate decays exponentially with time. Higgins (1976) made the exponent of this decay function dependent upon the available ground storage. However, once this available ground storage is filled, infiltration continues to approach its final rate.

CREAMS model. The CREAMS model (Knisel, 1980) calculates peak runoff rate by the following empirical formula:

$$q_p = 200.(DA^{0.70})(CS^{0.159})(LW^{-0.187})(Q^{0.917(DA^{0.0161})}) \qquad [II-7]$$

where

q_p = peak runoff rate in cfs

DA = drainage area in mi^2

CS = main channel slope in ft/mi

LW = watershed length to width ratio

Q = daily runoff volume in inches.

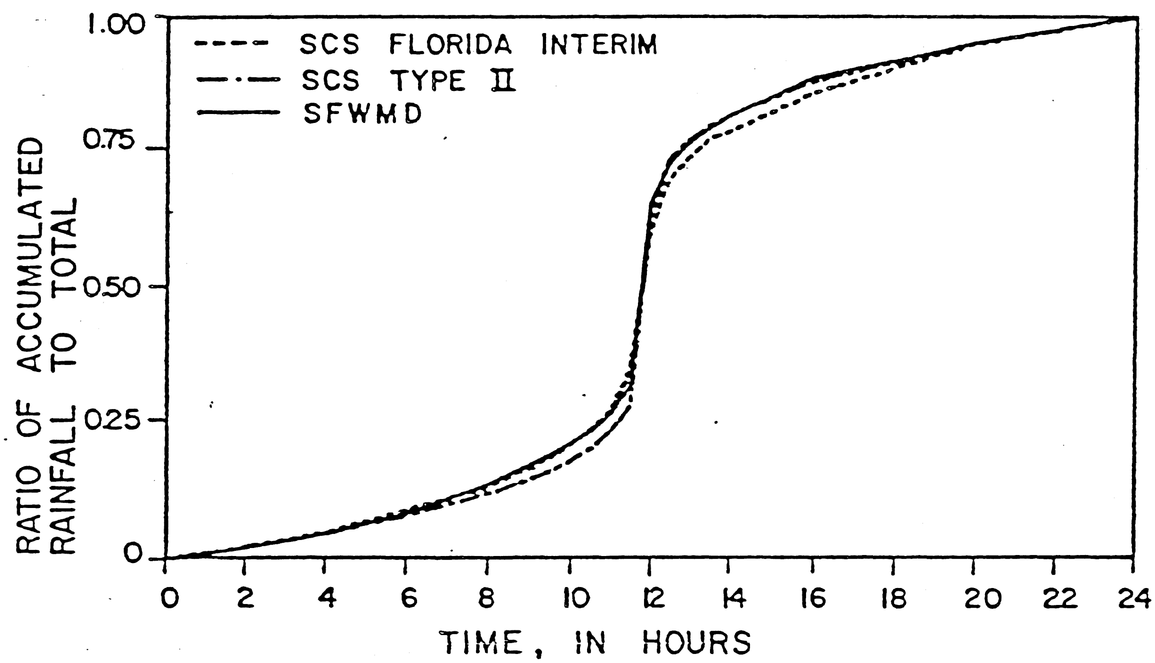


Figure II-4. Time-depth distributions for 24-hour design rainfall events.

This empirical formula was developed from data from 304 storms occurring on 56 watersheds in 14 states not including Florida (Smith and Williams, 1980).

Capece (1984) performed a regression of the CREAMS model formulation against measured data which yielded a modified version of Equation II-7:

$$q_p = 4.52(DA^{1.06})(CS^{0.77})(LW^{0.389})(Q^{0.87(DA^{-0.20})}) \qquad [II-8]$$

When reapplied to the data base, results were good. However, data were not available for use in an independent evaluation of the modified CREAMS equation.

SCS unit hydrograph method. The SCS unit hydrograph method of estimating peak runoff rates utilizes a triangular approximation of a runoff unit hydrograph (Figure II-5). Watershed and storm characteristics are used to estimate time parameters of the triangular hydrograph from which synthetic unit hydrographs can be created. The basic relationship can be written as:

$$q_p = \frac{(K')(A)(Q)}{T_p} \qquad [II-9]$$

where

q_p = peak runoff rate in cfs

A = area in mi^2

Q = rainfall excess depth in inches

T_p = time to peak in hours

K' = hydrograph slope and unit conversion factor.

Therefore, estimates of two parameters are required for synthesis of an SCS unit hydrograph. The standard estimate for K'(484) describes a hydrograph whose recession is 1.67 times as long as its time to peak. Mockus (USDA-SCS, 1972) noted that this K' value has been known to vary from 600 in steep terrain to 300 in flat swampy country. Welle et al. (1980) concluded that a value of 284 is more appropriate for the Delmarva peninsula, which includes Delaware and parts of Maryland. The watersheds examined were small with sandy souls and slopes in the range of 2 percent. The U.S. Army Corps of Engineers

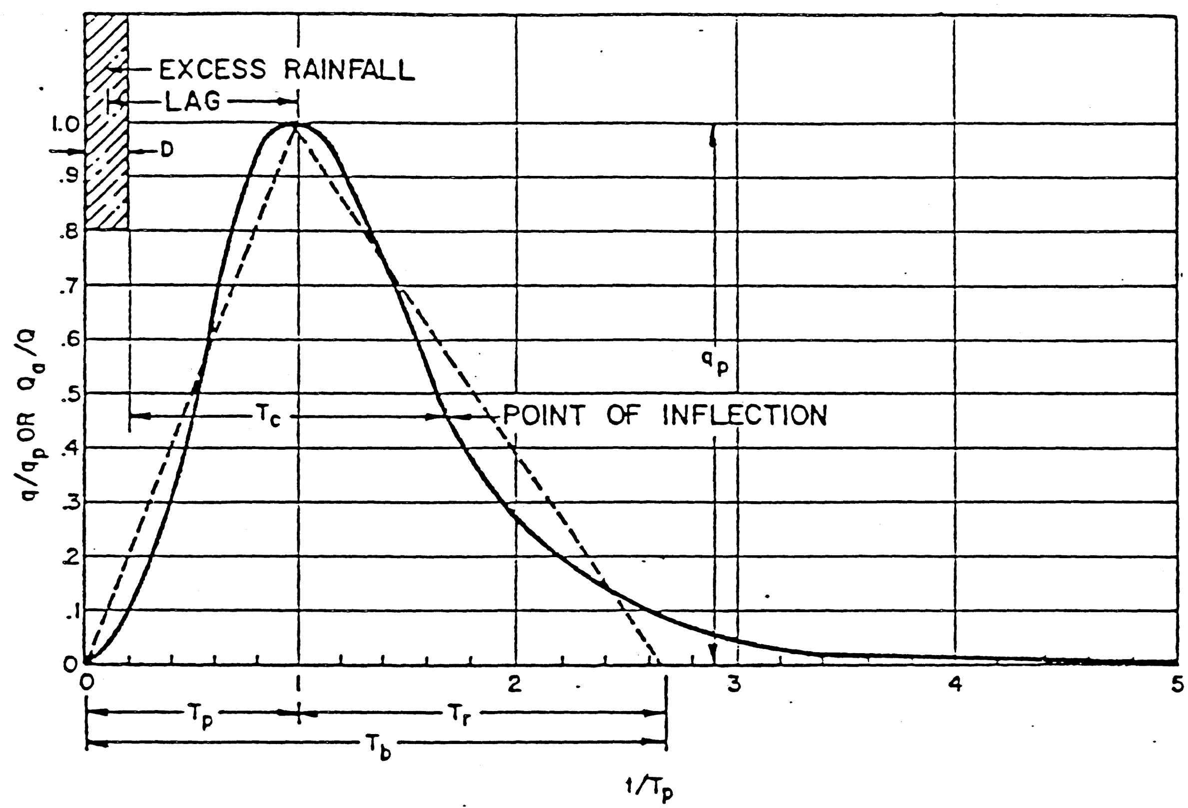


Figure II-5. SCS triangular unit hydrograph approximation and time parameter interpretations (USDA-SCS, 1972b).

(1955) studied records from several large watersheds in Central and South Florida (one of which was the entire Kissimmee River Basin) and determined an appropriate time factor for use in a similar peak discharge equation. Miller and Einhouse (1984) translated this factor into the SCS form arriving at a value of 256. Capece (1984) studied data from five small agricultural watersheds in Florida's flatwoods and determined that a value less than 100 was more appropriate.

The other time parameter in Equation II-9, T_p, is defined as

$$T_p = L + \frac{\Delta D}{2} \tag{II-10}$$

where

L = watershed time lag

ΔD = rainfall excess duration.

The SCS recommends using a duration not exceeding 20 percent of the time to peak. Lag can be calculated by the following equation:

$$L = \frac{\ell^{0.8}(S + 1)^{0.7}}{1900Y^{0.5}} \tag{II-11}$$

where

L = watershed lag

S = SCS watershed storage parameter

Y = average watershed slope in percent.

Alternatively, lag can be determined using total travel time. Capece (1984) found that, for Florida's flatwoods watersheds, an apparent correlation existed between T_p and percent wetlands (Figure II-6). This correlation served as the basis for the following equation:

$$L = 3.0 + 0.34(A^{0.11})(W + 1)^{0.71} \tag{II-12}$$

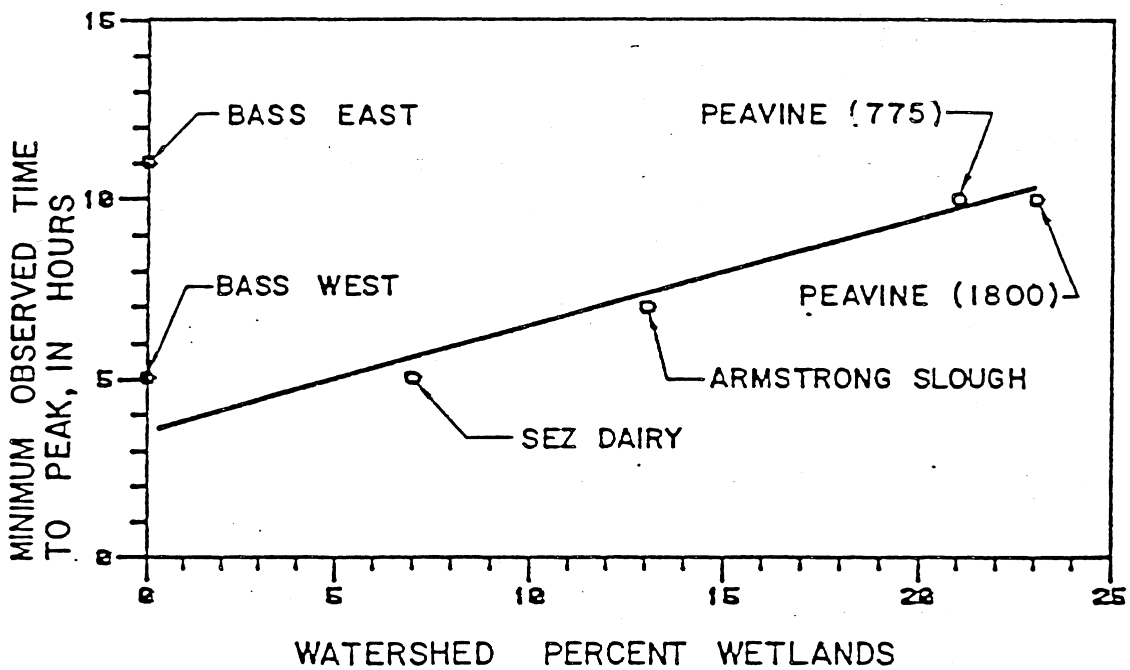


Figure II-6. Correlation between watershed minimum observed time to peak and watershed percent wetlands which served as the basis for the modified lag estimation equation.

where

L = watershed lag in hours

A = drainage area in acres

W = percent wetlands.

Given a triangular unit hydrograph tailored to a specific watershed and rainfall excess duration, a composite storm hydrograph can be developed by superposition of a series of incremental unit hydrographs of ΔD hours each within a storm event. Kent (1973) describes such a procedure. (Figure II-7).

Specific Stormwater Management Practices

Stormwater management practices are implemented to modify hydrograph shapes and to improve the quality of stormwater being discharged to receiving waters. Management practices are generally classified into two categories: structural and nonstructural. A short description of the most commonly used stormwater management practices follows. Included are urban and agricultural practices. Table II-1 summarizes the practices commonly used in Florida.

Structural Practices:

Structural practices are the primary means to achieve quantity and quality restrictions (Wanielista and Yousef, 1985). These are generally facilities designed to achieve a peak discharge reduction and pollution control. Other major areas are source modification and natural systems.

1. Off-Line Retention. This involves a diversion or baffle structure for diverting stormwater to an infiltration, percolation, or other treatment area. It is used to capture and retain the "first flush" of stormwater runoff. To be effective, these facilities must have the capacity for infiltration. The diverted water is not directly discharged to receiving waters. Off-line retention can be achieved by use of exfiltration systems (underground perforated pipe) or percolation basins.

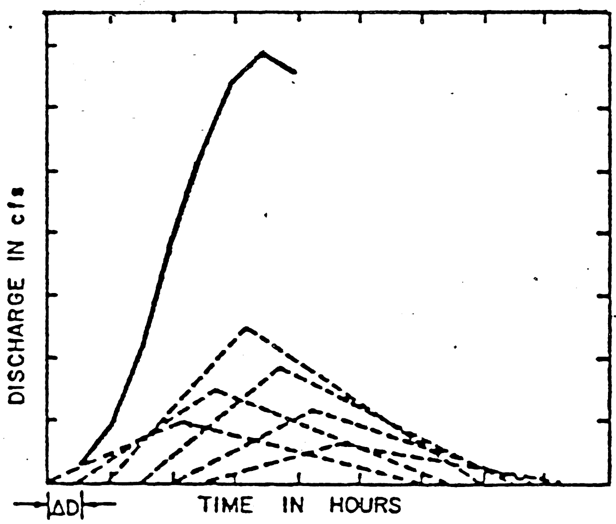


Figure II-7. Generation of a composite discharge hydrograph by the superposition of incremental unit hydrographs (Kent, 1973).

Table II-1. **CLASSIFICATION AND LISTING OF STORMWATER MANAGEMENT PRACTICES***
(Those More Commonly Used In Florida)

Structural (Control of Transport)		Nonstructural (Prevention of Generation and Accumulation)	
End-of-pipe	Retention Off-line On-line Detention Detention with Filtration Storage-Treatment	Surface Sanitation	Street Cleaning/Flushing Antilitter/Solid Waste Air Pollution Control
Source Modifications	Swales Rooftop Storage Parking Lot Storage Detention/Retention Porous Pavement	Chemical Use Control	Fertilizers/Pesticides Industrial Spillage Gasoline Stations Lead in Gasoline Highway Deicing
Natural Systems	Marsh Treatment Other Wetland Systems	Resource Planning	Computer Simulation I/I Studies
Sediment Control	Terracing Berming Contouring Sediment Trap Silt Fencing	Erosion Control	Seeding Sodding Mulching Road Stabilizers Stage Clearing

2. On-Line Retention. This generally involves ponds in which the residence time averages weeks to months. Pond depth is usually shallow (<4 feet) with the locations of the inlet-outlet structure such that short circuiting is prevented. Marsh areas can be created to aid in treating dissolved chemicals.

3. Detention. This is generally used to reduce peak runoff rates, but can provide some water quality improvement through reduction of particulates. These facilities are on-line ponds which slowly release stored waters. Therefore, holding times are relatively short (hours to days).

4. Detention with Filtration. This commonly involves bank and underdrain systems. These systems usually consist of a pipe and filter material (well graded sand). The pipes intercept, collect, and convey stormwater following infiltration through the soil and filter material (aggregate and filter fabric). The Florida Stormwater Rule for detention facilities (Chapter 17-25, Florida Administrative Code, 1985) requires that the first flush, typically the runoff from the first inch of rainfall, be both detained and filtered for systems that discharge to State waters.

5. Grassed Waterways and Swales. These are used to reduce runoff velocities, thereby enhancing infiltration and reducing pollutant and sediment loads to downstream waters. Swale blocks are often used to provide storage, allowing water greater time to infiltrate.

6. Wetlands Utilization. This practice involves routing stormwater through wetlands to reduce peak flow rates and provide water quality treatment. The Florida Department of Environmental Regulation has encouraged the use of wetlands as a stormwater management practice and has provided guidelines for their use in the Florida Stormwater Rule (Chapter 17-25, FAC, 1985). Rule requirements include not disrupting the normal range of water level fluctuation in the wetland and monitoring of wetlands stormwater discharge facilities.

Nonstructural Practices:

Nonstructural practices reduce the quantity of materials available for runoff and discharge to receiving waters. In order to provide acceptable pollutant reductions, they are generally used in combination with other practices.

1. Chemical-Use Controls. Included are fertilizer and pesticide use, industrial controls, gasoline station controls, reduction of lead in gasoline, and highway de-icing. Fertilization practices involve managing fertilizer use to reduce losses in runoff. Soil testing and proper timing of application are important. Pesticide control involves eliminating excessive pesticide use by proper application procedures and the use of biological pest control alternatives. Industrial process and gasoline station controls have become increasingly important because of the potential contamination of surface and ground waters.

2. Erosion/Sediment Controls. Include agricultural and construction practices designed to reduce soil erosion. Common practices are listed in Table II-1.

Flood Routing

Flood routing is used to convert an inflow hydrograph at the beginning of a structure or channel reach to an outflow hydrograph at the outlet or end of the reach. The purpose of flood routing in most engineering work is to learn what stages and rates of flow occur, without actually measuring them, at specific locations in streams or structures during the passage of floods (SCS NEH-4, Chapter 17, 1972). Water levels and flow rates are used in evaluating or designing a water control structure or project. In stormwater management, it is necessary to manage peak flow rates to protect property and control timing of the outflow hydrographs. Routing is used to predict the temporal and spatial variations of a flood wave as it traverses a channel reach or reservoir or can be used to predict the outflow hydrograph from a watershed subjected to a known amount of precipitation (Viessman et al., 1977). Routing techniques are classified into two categories: hydrologic routing and hydraulic routing.

Hydrologic routing uses the continuity equation with either an assumed or analytic relationship between storage and discharge within a system (i.e., ordinary differential equations are used to describe the system). Hydraulic routing techniques use both the equation of continuity and the conservation of momentum equation to describe spatial and temporal variations (i.e., partial differential equations are necessary for describing the system).

Hydrologic routing techniques are simple to use but usually require that the parameters be fitted to observed data. They do not adequately evaluate backwater effects or the effect of surges. Hydraulic routing techniques more adequately describe the dynamics of flow and use physically based parameters. However, they generally require large numbers of inputs and are usually difficult to solve. Linsley et al. (1982), Viessman et al. (1977), and Henderson (1966) discuss details of these techniques.

Hydrologic Routing:

In hydrologic routing, the continuity equation is written based on a mass balance of the channel reach or reservoir. It may be expressed as:

$$I - O = \frac{dS}{dt} \qquad [II-13]$$

and in finite difference form

$$S_2 + \frac{O_2}{2}\Delta t = S_1 - \frac{O_1}{2}\Delta t + \frac{I_1 + I_2}{2}\Delta t \qquad [II-14]$$

In these equations, the subscripts 1 and 2 refer to conditions at the beginning and end of a time interval, respectively; I represents the inflow rate; O represents the outflow rate; S is the storage; and Δt is the time interval. An additional relationship between storage and outflow must be defined in order to solve the equation. Several hydrologic routing methods are presented here.

<u>Modified Puls or storage indication method</u>. The Puls method is generally used in reservoir routing. The method used a finite difference approximation for the continuity equation. In Equation II-14, the only unknowns are on the left-hand side. From the actual relationship between storage and outflow a table or graph of O vs $S + \frac{\Delta t}{2} O$ is constructed. At each time step, the right-hand side of Equation II-14 is calculated and set equal to the left-hand side; O and S are then found from the O vs $S + \frac{\Delta t}{2} O$ and S vs O relationships. This is repeated at each time step.

A defect of the Puls method is that there is no rule for selecting the proper size of routing interval (SCS, 1972). Due to mathematical instability, usually caused by high inflow rates causing large changes in stage over one routing time step, rising portions of the outflow hydrograph can be distorted and negative outflow rates may occur during recession periods if O_2 is greater than $(S + \frac{\Delta t}{2} O_2)$. Therefore, it is important when using this method that a small enough time interval is selected so that numerical instability problems do not interfere with the accuracy of routing computations.

Muskingum method. The Muskingum method is one of the most commonly used channel routing techniques. In the Muskingum method, a relationship between storage and outflow discharge is obtained by dividing the storage in the channel reach into two components, prism and wedge storage (Figure II-8). Prism storage is conceptualized as a function of outflow discharge (i.e., depth at point 2) and wedge storage is assumed to be a function of the difference between inflow and outflow discharge (i.e., depth associated with I-O at point 1). Total storage within the reach is the sum of both prism and wedge storage. By assuming a linear relationship between storage and discharge ($S_T = K\,Q$), total storage within the reach can be defined as:

Total Storage = Prism Storage + Wedge Storage

or

$$S_T = KO + KX(I - O) \qquad [II\text{-}15]$$

or

$$S_T = K[XI + (1 - X)O] \qquad [II\text{-}16]$$

where S_T, I, and O are defined previously, K is the storage time constant for the reach and X is a shape parameter which weights the importance of inflow and outflow discharge.

Equations II-15 and II-16 can be combined to form the Muskingum routing equation:

$$O_2 = C_0 I_2 + C_1 I_1 + C_2 O_1 \qquad [II\text{-}17]$$

where

$$C_1 = \frac{KX + 0.5\ \Delta t}{K(1 - X) + 0.5\ \Delta t} \qquad [II\text{-}18]$$

$$C_2 = \frac{-KX + 0.5\ \Delta t}{K(1 - X) + 0.5\ \Delta t} \qquad [II\text{-}19]$$

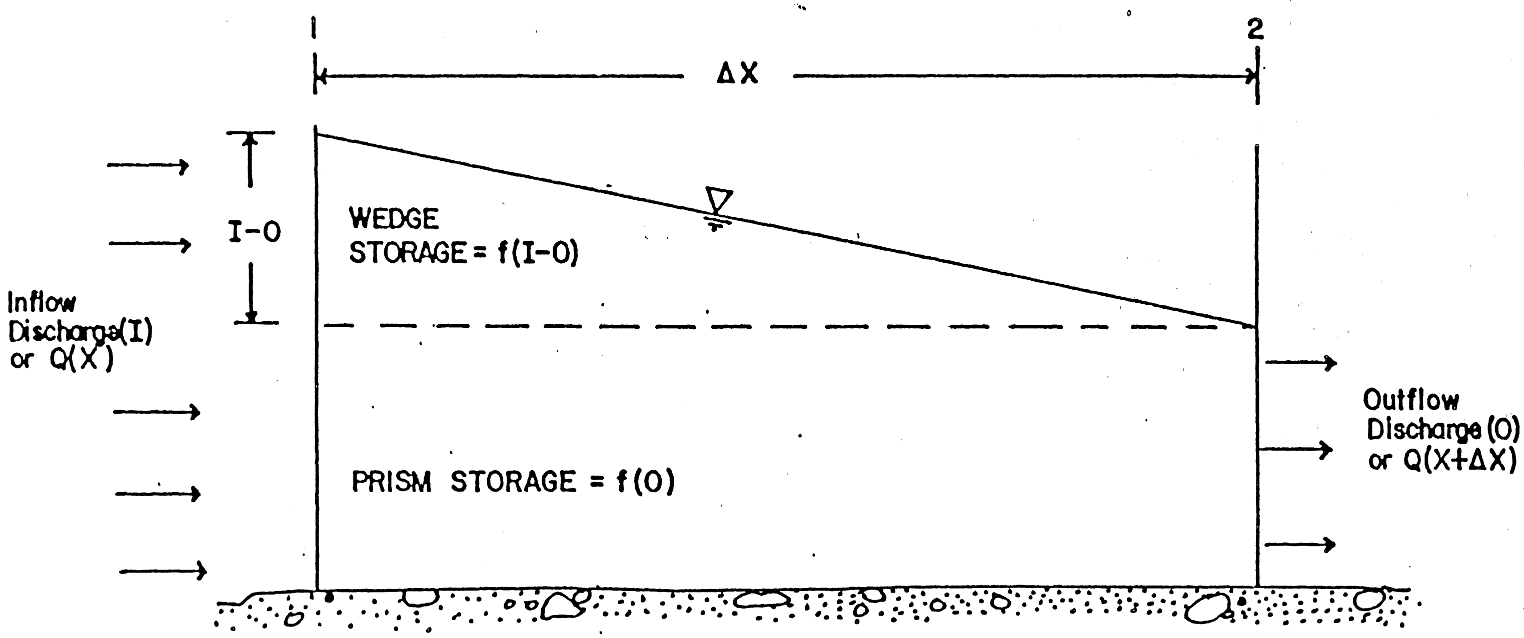


Figure II-8. *Muskingum's Routing Storage Relationships*

$$C_3 = \frac{K(1 - X) - 0.5\ t}{K(1 - X) + 0.5\ t} \quad [II-20]$$

$$C_1 + C_2 + C_3 = 1 \quad [II-21]$$

Values of K and X are commonly estimated using K = travel time of flood wave in the reach and the average value of X = 0.2. For gaged watersheds, K and X can be determined by 1) least squares method, 2) method of moments, 3) method of cumulents, 4) graphical method, and 5) direct method (Singh and McCann, 1979). The graphical method is generally used. It involves a plot of (XI + (1 - X)O) against storage. The value of X is varied until a single line can represent the data. The slope of this line is used to determine K (see Linsley et al., 1975; Viessman et al., 1977). This is shown in Figure II-9.

For ungaged watersheds estimates of K and X can be obtained by manipulating Muskingum's equation (Equation II-16) into a form similar to the diffusion model (Cunge, 1969; Koussis, 1978). The parameters K and X can then be defined based on hydraulic properties.

Linear reservoir method. A linear reservoir is a conceptual reservoir in which the storage, S, is directly proportional to the outflow, Q, or:

$$S = K\ Q \quad [II-22]$$

The proportionality constant, K, is known as the storage coefficient. Substituting Equation II-22 in Equation II-13,:

$$I - Q = \frac{KdQ}{dt} \quad [II-23]$$

or

$$\frac{dQ}{dt} + \frac{Q}{K} = \frac{I}{K} \quad [II-24]$$

If I is known, this can be solved analytically (see Henderson, 1966).

SCS Convex method. The Convex method (SCS, 1972) involves only inflow-outflow hydrograph relationships. The continuity equation is not directly involved. Therefore, close adherence to procedures recommended by the SCS is necessary. The routing equation in the Convex method is:

$$O_2 = (1 - C)O_1 + CI_1 \quad [II-25]$$

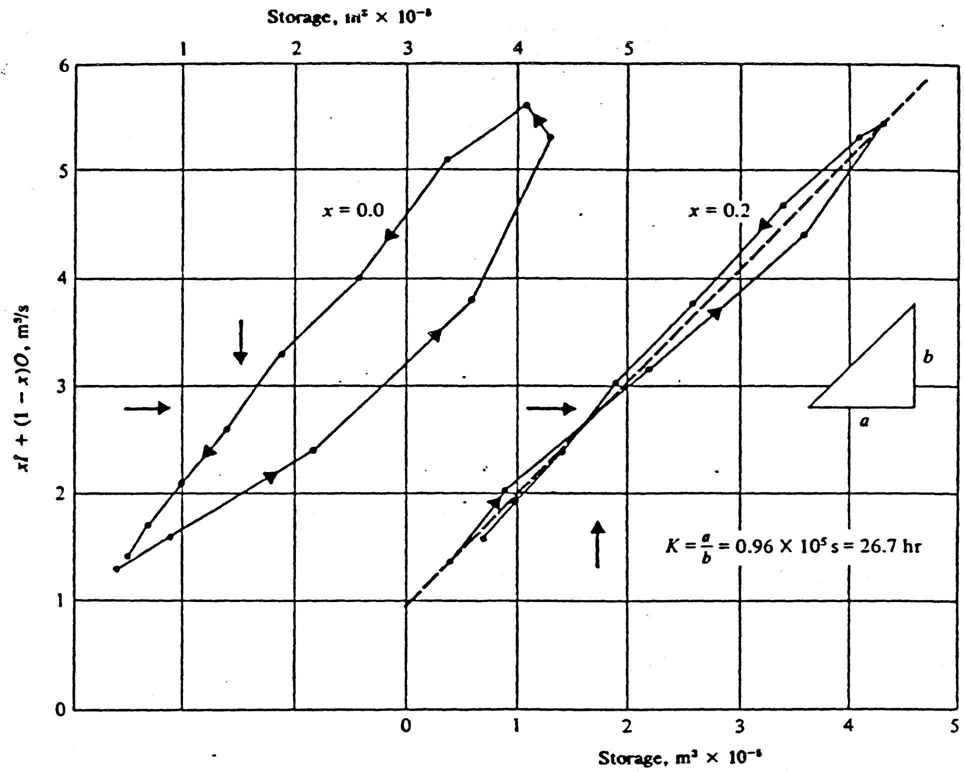


Figure II-9. Graphical Determination of K

where

C is a parameter such that $0 \leq C \leq 1.0$; C can be estimated from:

$$C = V/(1.7 + V) \qquad [II-26]$$

where V is the average flow velocity in the reach; V may be computed at bankful discharge at a flow equal to 75 percent of the peak flow, or at some other appropriate value. The C value may also be approximated from the X in the Muskingum method as $C \simeq 2_X$ if an appropriate X is available.

The proper routing interval to use with the Convex method is:

$$\Delta t = C\ K \qquad [II-27]$$

where K is a parameter equal to the reach travel time. This method of computing Wt generally results in an inconvenient time interval. A more convenient time interval can be calculated from:

$$C^* = 1 - (1 - C)^{\Delta t^* / \Delta t} \qquad [II-28]$$

where Wt is from Equation II-27 and Wt* is the desired time interval. The ratio $\Delta t^*/\Delta t$ should be kept as near unity as possible. Note that the ratio of $\Delta t^*/\Delta t$ can be varied by changing the reach length as well as the routing interval.

Modified Att-Kin method. The Modified Att-Kin (Attenuation-Kinematic) procedure (SCS, 1982) is based on the Att-Kin procedure (SCS, 1979) which was modified to conform the structure of the existing TR-20 computer program (SCS, 1982). The Att-Kin procedure is based on storage and kinematic models to reflect the reservoir and translation effects on natural floodwaves, respectively.

The method uses the routing equation as contained in the Convex method (Equation II-25). The continuity equation is written as:

$$I_1 - \left(\frac{O_2 + O_1}{2}\right) = \frac{\Delta S}{\Delta t} \qquad [II-29]$$

where I_1 is the average inflow over the time interval Wt and $\left(\frac{O_2 + O_1}{2}\right)$ is the average outflow over the same time interval.

Storage, S, is defined as:

$$S = KO \qquad [II-30]$$

where K approximates the slope of the storage-outflow curve at peak outflow discharge (Figure II-10).

Equation II-29 becomes:

$$I_1 - \left(\frac{O_2 + O_1}{2}\right) = K\left(\frac{O_2 - O_1}{\Delta t}\right) \qquad [II-31]$$

Expressing Equation II-31 in the form of Equation II-25 yields an equation for C,:

$$C = \left(\frac{2\Delta t}{2K + \Delta t}\right) \qquad [II-32]$$

The value of K is used to solve for C; C is not allowed to exceed 1. The time difference between the inflow peak and the outflow peak is Δt_{ps}. The value of Δt_{ps} is a multiple of the main time increment used in the reach routing. The minimum value of Δt_{ps} is the main time increment. The resulting hydrograph is then positioned in time so the difference between the time to peak of inflow and outflow is:

$$\Delta t_p = \frac{S_{pi} - S_{po}}{Q_{pi} - Q_{po}}\left(\frac{1}{3600}\right) \qquad [II-33]$$

where

S_{pi} = valley storage associated with the inflow peak, ft^3

S_{po} = maximum valley storage in the reach during the passage of and assumed coincident with the outflow peak, ft^3

Q_{pi} = peak of the inflow hydrograph including base flow, cfs

Q_{po} = peak of the outflow hydrograph including base flow.

Δt_p = elapsed time between the inflow and outflow peaks, hrs.

The equation for Δt_p is developed from the kinematic travel time assuming that peak discharge varies linearly with time during its movement through the reach. The time to peak of the kinematic routed hydrograph (Δt_p) is used to position the outflow hydrograph if $\Delta t_p > \Delta t_{ps}$.

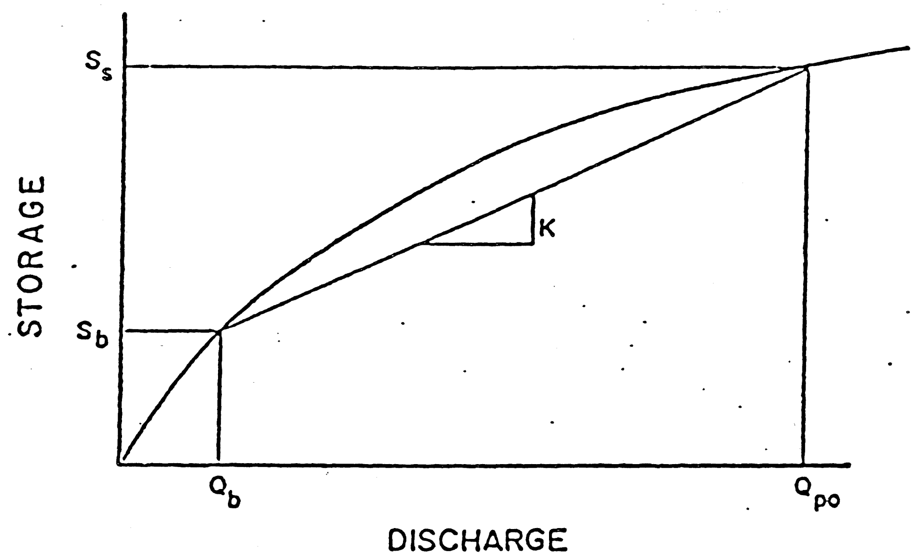


Figure II-10. Storage-outflow slope used in the Modified Att-Kin routing

Hydraulic Routing:

Hydraulic flow routing is based on the continuity equation and the conservation of momentum equation. The continuity and conservation of force-momentum equations are:

$$\frac{\partial Q}{\partial X} + \frac{\partial A}{\partial t} = q_\ell \qquad [II\text{-}34]$$

and

$$\underbrace{\frac{\partial V}{\partial t} + V\frac{\partial V}{\partial X} + \frac{Vq_\ell}{A}}_{\text{Inertia}} + \underbrace{g\frac{\partial Y}{\partial X}}_{\text{Pressure}} = \underbrace{g\,(S_0}_{\text{Gravity}} \underbrace{-\ S_F)}_{\text{Friction}} \qquad [II\text{-}35]$$

where

Q = discharge (L^3/T)

A = cross-sectional area

q_P = lateral inflow per channel length (L^2/T)

V = velocity (L/T)

t = time

S_0 = channel slope

S_F = friction slope

X = distance (L)

Y = depth (L)

g = acceleration of gravity (L^2/T).

There are basically three different hydraulic routing techniques used: dynamic wave, diffusion wave, and kinematic wave. The models differ depending on the assumptions used to evaluate the momentum equation. The dynamic wave model uses the momentum equation in its complete form. The diffusion wave model assumes that the inertia terms can be ignored without appreciable error. Therefore, its flow rate function becomes:

$$\frac{\partial Y}{\partial X} = S_0 - S_F \qquad [II\text{-}36]$$

The kinematic wave model neglects both the inertia and pressure terms. Hence the flow rate function becomes:

$$S_0 = S_F \qquad [II\text{-}37]$$

The kinematic wave solution is given by a rating curve relationship:

$$q = a\ y^b \qquad [II\text{-}38]$$

The assumption given in Equation II-37 can also be developed from this viewpoint. Using Manning's equation, nonuniform flow is calculated by:

$$Q = A \frac{1}{n} R^{2/3}\ S_p^{\ 1/2} \qquad [II\text{-}39]$$

Uniform, or normal flow, is calculated by:

$$Q_n = A \frac{1}{n} R^{2/3}\ S_0^{\ 1/2} \qquad [II\text{-}40]$$

Therefore,

$$Q = Q_n \sqrt{S_F/S_0} \qquad [II\text{-}41]$$

For the kinematic wave model (i.e., $S_0 = S_F$), the nonuniform discharge is equal to the normal discharge and satisfies the rating curve relationship in Equation II-38.

Equation II-41 can be used to highlight the differences between the hydraulic models. If lateral inflows are neglected, Equation II-41 can be written as:

$$S_F = S_0 - \frac{\partial Y}{\partial X} - \frac{V}{g}\frac{\partial V}{\partial X} - \frac{1}{g}\frac{\partial V}{\partial t} \qquad [II\text{-}42]$$

By substituting Equation II-42 into Equation II-41, the differences can be illustrated as follows:

$$Q = Q_n \sqrt{1 - \frac{1}{S_0}\frac{\partial Y}{\partial X} - \frac{V}{gS_0}\frac{\partial V}{\partial X} - \frac{1}{gS_0}\frac{\partial V}{\partial t}} \qquad [II\text{-}43]$$

Kinematic wave T

Diffusion wave T

Dynamic wave T

Chow (1959), Henderson (1966), or Viessman et al. (1977) contain details of solution techniques. Hydraulic routing models are generally solved by the Method of Characteristics or by finite difference methods. The reader is referred to the above-mentioned references for further details.

Stormwater Management Design Constraints

Before a site can be developed for agriculture, required permits for construction must be obtained from the appropriate governmental bodies. In order to obtain the required permits, applicants must demonstrate that their project is in compliance with the established regulatory criteria. The following paragraphs will summarize the criteria, or constraints, within which a design engineer must operate when designing a stormwater management system for a particular site. The following criteria were taken from the South Florida Water Management District's Permit Information Manual, Vol. IV (1986). It should be noted that criteria in other areas of Florida are similar and that the criteria from South Florida Water Management District (SFWMD) are being used for illustrative purposes. It should also be noted that only the criteria deemed applicable to agricultural projects are presented here. The criteria can be grouped into six areas: environmentally sensitive areas, floodplain encroachment, off-site discharge, on-site storage, water quality, and low flows and groundwater maintenance.

Wetland Areas:

This generally refers to the soils and vegetation present on the site. These areas are to be preserved or mitigated if disturbed. In certain instances, smaller isolated areas may be disturbed or "traded off" for larger areas which may be comprised of more valuable uplands and wetlands. These areas can potentially be incorporated into the stormwater management system as a storage body. Designation of an environmentally sensitive wetland area is generally determined by the regulatory agency. The soils, vegetation, wildlife, and hdyrologic function of the area are factors in this determination.

Floodplain Encroachment:

The SFWMD <u>Permit Information Manual</u>, Vol. IV, states that no net encroachment into the floodplain between the average wet season water table and that encompassed by the 100-year, 3-day storm event, which will adversely affect the existing rights of others, will be allowed. There are two aspects to the question of flood plain encroachment: storage reduction and flow interference. Therefore, the storage and flow characteristics of the floodplain must be maintained. It is sometimes difficult to identify floodplain areas in the flatwoods region. In many cases, considerable engineering judgement is required.

Off-Site Discharge:

Off-site discharge is limited to amounts which will not cause additional adverse off-site impacts. These amounts are generally determined in two ways. The first involves a comparison of the peak discharges from the site in its natural condition and in its developed condition with the goal in the developed condition being to match the peak discharge that occurred in the site's natural state. A 25-year, 3-day storm event is used in this analysis. It is assumed that the resulting stages will also match. Occasionally, the duration of high stages is a concern which must be addressed by the design engineer. The second way to determine the limiting off-site discharge amount is to use an amount specified in SFWMD criteria. Appendix II-3 gives allowable discharge formulas for various basins within the District. Flows from areas upstream of the site are to be routed through, or around, the project site. It is assumed for design purposes that the water-table depth antecedent to the storm event is at the average wet season level.

On-Site Storage:

On-site storage is generally necessary; if not for quantity management, then for quality management as will be discussed later. The SFWMD prefers separately contained storage areas which are fed by pumps or gravity if topography allows and which discharge by gravity. Unless a detailed dam structural safety analysis of above-ground dikes is available, the maximum above-grade water depth which can be stored is 4.0 feet. Freeboard shall be equal to the water depth but no less than 2.0 feet and not more than 3.0 feet above the depth of the stored routed design storm. (This is quoted from the 1986 SFWMD manual and is not clear.) Certain project-specific factors may affect the recommended criteria. Pump filled impoundments require a separate overflow structure which conducts flows back into the property. A shaft spillway, or weir structure, may be used for this. Design recommendations are that the weir crest be set at the peak elevation of the routed 25-year, 3-day storm and that the weir crest length be adequate to pass the difference between the routed (if any) pumped inflow hydrograph plus the 100-year, 3-day rainfall on the reservoir, minus the routed outflow through the control structure, with a freeboard on the dikes of at least 1 foot.

Water Quality:

The basic criterion is the detention of the first inch of runoff or the runoff from a 2.5 inch rainfall, whichever is greater. For agricultural sites, the 1 inch is generally applicable. This volume must be detained and allowed to release within 5 days with approximately one-half inch of detention volume discharged on the first day. The control, or bleeddown, structure should be of a "V" or circular shaped configuration. The District prefers that this detention volume be held in a separately contained area rather than in the internal field water management system due to potentially being incompatible with on-field flood protection goals.

Low Flows and Groundwater Maintenance:

Since the District requires that off-site upstream runoff be passed around, or through, a project, low flows are generally maintained. The District does require that a project not alter water tables in a manner that would cause off-site problems. It is required that projects not control internal water levels deeper than 6 feet below ground level. In more hilly areas, this will require internal "step down" control structures.

The Effectiveness of Stormwater Management

The concept of stormwater management in relation to the overall water resource is relatively new. The approach until the early 1970s relied on swales, curb and gutter, inlets, storm sewers, and channels to carry away flow as quickly as possible (Urbonas and Tucker, 1984). This approach has been modified in recent years by the introduction of detention storage to hold back runoff and release it at controlled rates. Jones and Jones (1982) state that because of the simplicity and attractiveness of the detention concept, combined with lack of awareness or appreciation of the complex interrelationships which influence performance, detention storage is widely misapplied. McCuen (1974) suggests that detention storage may increase flooding problems rather than reduce them. Because of the potentially harmful effects which may occur, a designer needs to look at the off-site impacts his project may have. Hawley et al. (1982) and Mynear and Haan (1980) have suggested methods for evaluating a system of detention basins for effectiveness and potential off-site effects.

The requirement that postdevelopment peak discharge rates not exceed predevelopment peak discharge levels is sometimes inadequate for effective stormwater management. The timing and duration of the developed peak discharge rate can produce undesirable effects if significantly different from the undeveloped case. McCuen (1974) pointed out that if land near the outfall of a watershed were being developed and provided with detention structures, the delayed flood peak could reach the outfall at the same time as the flood peak from further upstream and cause a greater peak at the outfall than would occur with detention. Smiley and Haan (1976) considered this problem and showed examples of detention structure placement being detrimental using SCS-TR20 (1965). Related to the above problem, Hardt and Burges (1976) showed that restricting outflow to predevelopment rates could achieve a composite peak flow rate that would equal the preurbanization flow but would run for a much greater duration, due to greater total runoff volume, resulting in potentially undesirable effects.

The particular recurrence interval of a storm event used in the design process can have a bearing on stormwater management effectiveness. Urbonas and Tucker (1984) found that random on-site detention has the potential for being reasonably effective in controlling some larger storm flows along major drainage ways but may not be effective for controlling frequently occurring flows such as 2-year storm in the Denver region. They also found that system effectiveness in regards to controlling flow rates along major drainage ways is limited only to events of the same design recurrence frequency for which the ponds are designed and that ponds designed to control the peak flow of two separate recurrence frequencies appear to be effective in controlling flow rates along major drainage ways for a range of flows and the two individual design storms. While the authors warn against extrapolating these conclusions to other regions, it would suggest that a design based on multiple storm recurrence intervals would be beneficial. Jones and Jones (1982) state that facilities designed on the basis of relatively frequent events usually have little attenuating effect upon runoffs from infrequent extreme events. They suggest that design of detention-pond outlet works often should have a multi-probability basis: 1) for frequent low flow conditions, 2) for the detention design discharge condition, and 3) for the extreme runoff (emergency spillway) condition. Another solution has been suggested by Sandvik (1985) which involves the construction of a "proportional" weir which is designed to control peak flows from multiple return-period storms. The structure is more sophisticated than typical weir structures and is somewhat expensive. However, it may be feasible to use in large projects.

Historically, stormwater management systems have been designed for discharge quantity control only. Guidelines for stormwater quality control have only recently begun to be established (NJDEP, 1981; DRCC, 1979). Methods for controlling pollutants generally involve controlling the pollutant transport at its source. These methods are commonly referred to as Best Management Practices or "BMPS." Now, detention basin design procedures for the purpose of both stormwater quantity and quality control are beginning to be formulated (Whipple, 1979; McCuen, 1980). Modeling approaches are also being developed (Ferrara and Hildick-Smith, 1982). Designing for these dual purposes involves different criteria for each purpose (Davis et al., 1978). While riser characteristics are important for stormwater flow rate control, the flow length and detention time are important in water pollution control. As loading functions for detention facilities become better defined, dual purpose design procedures will improve.

Trends, such as increased use of computers to solve drainage problems, consideration of both quantity and quality control, increased efforts to collect good data of appropriate spatial and temporal resolution should result in more effective systems (Colyer and Yen, 1983). Linsley and Crawford (1974) have suggested the use of continuous simulation models in urban hydrology. Many excellent models, such as SWMM (1971), are available today. The use of such models will increase as more data become available and simulation costs decrease.

Most of the research regarding stormwater management has taken place in an urban setting. Some research relating to agricultural systems has been performed. Tai (1975) performed a study of an agricultural reservoir located in St. Lucie County, Florida. While the primary purpose of this reservoir was to augment irrigation needs, Tai (1975) concluded that, if managed properly, the reservoir could effectively control peak flow rates from the site.

A Design Model for Agriculture in Florida's Flatwoods

In order to analyze agricultural stormwater systems, a model was needed which simulates the hydrology of the area well, accounting for all components of the water budget, and is capable of routing runoff through the system. The model which was developed for this purpose will be referred to as CRWT-HYDRO. The model uses the CREAMS-WT model (Heatwole, 1986) as the hydrologic component. A routine to generate runoff hydrographs was incorporated and linked to a multibasin reservoir-channel routing program. A description of the model follows.

Model Structure:

CRWT-HYDRO was developed with the goal being to maintain the original structure of CREAMS-WT. The model remains field scale with the major modification being the ability to compute runoff hydrographs from field subareas. Therefore, the model is still applicable to field-sized areas with similar hydrologic characteristics.

In CRWT-HYDRO, the field is divided into a sequence of field subareas, channels, and reservoirs. At any point in the field where the user wishes to combine hydrographs, a structure must be designated. This is done in an input file separate from CREAMS-WT. Up to 10 subareas can be modeled. This can be increased if computer memory storage is available. The flood routing program can handle any combination of 10 reservoirs and channels.

When the user wishes to combine subarea hydrographs, "null" structures in the form of channels must be specified in that the program does not combine runoff hydrographs unless they are routed in some manner. Therefore, a requirement of the model is that all subarea hydrographs must be routed through a corresponding structure.

The input data for the hydrology portion of the model are the same as for CREAMS-WT with a few exceptions. The additional input parameters include drainage area, lag, desired rainfall distribution, and peak rate factor. The input data for the flood routing portion is placed in a separate file so that the file for CREAMS-WT will remain basically unchanged. Input parameters include the number of channels, subareas, and reservoirs; Muskingum's K and Muskingum's X for each channel; stage and storage data for each reservoir; discharge structure type and pertinent dimensions and control elevations; and how hydrographs are to be combined. A requirement of CRWT-HYDRO is that the input data begin at the farthest upstream structure and then proceed in numerical order downstream to the field outlet.

Rainfall Extraction

CRWT-HYDRO uses CREAMS-WT to compute runoff. CREAMS-WT incorporates revised SCS procedures adapted to flatwoods, accounts for all components of the water budget, and provides continuous simulation of water-table movement so that better estimates of conditions antecedent to storm events can be made. CREAMS-WT also allows the user to utilize real daily rainfall data or to specify a particular design storm. CREAMS-WT also contains a chemical component so that nutrient transport could be simulated.

Overland Flow

The overland flow component of CRWT-HYDRO is predicted using SCS unit hydrograph techniques. This technique was selected because 1) it can be easily adapted to the revised SCS procedures contained in CREAMS-WT, 2) it can easily distinguish differences in subareas, and 3) it is capable of handling multi-day (complex) events. The user has the option of specifying what peak rate factor (K') to use. Capece (1984) found that a K' less than 100 was appropriate for flatwoods. However, this value could be estimated based on the user's judgement or some other method (McCuen and Bondelid, 1983). CRWT-HYDRO uses a value of 100 for K'.

The time to peak of the unit hydrograph is defined in CRWT-HYDRO as:

$$t_p = 0.6\ t_c + D/2 \qquad [II-44]$$

where D is the convolution time interval and t_c is the time of concentration. t_c is estimated by the following equation.

$$t_c = L/0.6 \qquad [II-45]$$

where L = watershed lag. Lag is determined from the equation developed by Capece (1984) which relates lag time to drainage area and percent wetlands. Lag is defined as

$$L = 3.0 + 0.34(A^{0.11})(W+1)^{0.71} \qquad [II-46]$$

where

L = watershed lag in hours,

A = drainage area in acres, and

W = percent wetlands.

CRWT-HYDRO allows the user to vary the convolution interval.

For basins where t_c is short, the model will simulate overland flow using an instantaneous hydrograph.

<u>Reservoir Routing</u>

CRWT-HYDRO uses the Puls method to route flows through reservoirs. The Puls method is well known and is generally used in reservoir routing. CRWT-HYDRO solves equation [II-14] in the following manner. Equation [II-14] can be written as:

$$S_2 = \frac{\Delta t}{2}[(I_1+I_2) - (O_1+O_2)] + S_1 \qquad [II-47]$$

The unknowns in this equation are S_2 and O_2. Both S and O are functions of the stage (H); therefore, S and O are functionally related, which allows a unique solution for O_2. The equation is solved iteratively by the following procedure:

(1) Assume that $O_2 = O_1$

(2) Compute S_2 from Equation II-47

(3) Determine H_2 from S_2 from the stage storage curve

(4) Using H_2, determine O_2 from the appropriate structure discharge equation

(5) Repeat steps 2-4 until O_2 converges.

Two iterations will generally be sufficient to make solutions converge.

The user can select from seven different structures. They are:

1) weir
2) submerged weir
3) circular orifice
4) triangular orifice
5) V-notch weir
6) combination weir - V-notch weir
7) pump.

Channel Routing

CRWT-HYDRO uses the Muskingum method for routing flows through channels. This method is one of the most commonly used methods for routing through channels and is computationally efficient. The routing parameters K and X can be determined by using the method described in Cunge (1969) and Koussis (1978). Using this method, the parameters K and X can be defined based on hydraulic properties. A derivation of this method using different symbols follows (i.e., X becomes Θ).

The inflow and outflow are written as a discharge located at X ($I = Q(X,t)$) and $X+\Delta X$ ($O = Q(X+\Delta X, t)$), respectively. Therefore, the continuity equation (Equation II-13) can also be written as:

$$\frac{dS_T}{dt} = Q(X,t) - Q(X+\Delta t) \qquad [II-48]$$

By using a Taylor series expansion about X, $Q(X+\Delta X, t)$ can be approximated as:

$$Q(X+\Delta X,t) = Q(X,t) + \Delta X \frac{\partial Q(X,t)}{\partial X} + \frac{\Delta X^2}{2} \frac{\partial^2 Q(X,t)}{\partial X^2} \qquad [II-49]$$

where the higher order terms have been neglected. Substituting this expression into Equation II-48 yields:

$$\frac{dS_T}{dt} = -\Delta X \frac{\partial Q(X,t)}{\partial X} - \frac{\Delta X^2}{2} \frac{\partial^2 Q(X,t)}{\partial X^2} \qquad \text{[II-50]}$$

By substituting Q(X,t) and Q(X+ΔX, t) into Muskingum's storage equation (Equation II-15) for inflow and outflow discharges, and by differentiating with respect to time yields:

$$\frac{\partial S_T}{\partial t} = K\Theta \frac{\partial Q(X,t)}{\partial t} + K(1-\Theta) \frac{\partial Q(X+\Delta X,t)}{\partial t} \qquad \text{[II-51]}$$

Substituting Equation II-49 into this expression and neglecting third order terms produces:

$$\frac{\partial S_T}{\partial t} = K \frac{\partial Q(X,t)}{\partial t} + K(1-\Theta)\ \Delta X \frac{\partial^2 Q(X,t)}{\partial X \partial t} \qquad \text{[II-52]}$$

From the kinematic wave equation ($\frac{\partial Q}{\partial t} + C \frac{\partial Q}{\partial X} = 0$), the following relationship may be used:

$$\frac{\partial Q}{\partial t} = - C \frac{\partial Q}{\partial X} \qquad \text{[II-53]}$$

where C is the kinematic wave velocity. This is substituted into Equation II-52 to remove the mixed derivative term to yield:

$$\frac{\partial S_T}{\partial t} = K \frac{\partial Q}{\partial t} - CK(1-\Theta)\ \Delta X \frac{\partial^2 Q}{\partial X^2} \qquad \text{[II-54]}$$

For a given stream reach, storage over the reach length of WX is dependent on time only. Therefore, the partial derivative of storage in Equation II-54 may be set equal to the total derivative. Equations II-50 and II-54 can now be set equal to each other yielding:

$$-\Delta X \frac{\partial Q}{\partial X} - \frac{\Delta X^2}{2} \frac{\partial^2 Q}{\partial X^2} = K \frac{\partial Q}{\partial t} - CK(1-\Theta)\ \Delta X \frac{\partial^2 Q}{\partial X^2} \qquad \text{[II-55]}$$

Rearranging terms,

$$\frac{\partial Q}{\partial t} + \frac{\Delta X}{K} \frac{\partial Q}{\partial X} = [\Delta X(1-\Theta)C - \frac{\Delta X^2}{2K}] \frac{\partial^2 Q}{\partial X^2} \qquad \text{[II-56]}$$

which is a form of the advective diffusion equation. The diffusion model can be written as:

$$\frac{\partial Q}{\partial t} + C \frac{\partial Q}{\partial X} = D \frac{\partial^2 Q}{\partial X^2} \qquad \text{[II-57]}$$

where D is the numerical diffusivity.

These equations become equivalent if C and D are defined by:

$$c = \Delta X/K \qquad [II-58]$$

and

$$D = \Delta X(1-\Theta)C - \frac{\Delta X^2}{2K} \qquad [II-59]$$

From Equations II-58 and II-59, the physical properties can be assigned to K and R. The kinematic wave speed, c, has been defined from Manning's equation as 5/3 of the velocity:

$$C = \frac{5}{3}(\frac{1}{n} R^{2/3} S^{1/2}) = \frac{5}{3} V \qquad [II-60]$$

Therefore, for the kinematic model K can be evaluated as:

$$K = \frac{3}{5} \frac{\Delta X}{V} \qquad [II-61]$$

For a wide rectangular channel, D has been defined as:

$$D = \frac{Q}{2S_F w} \qquad [II-62]$$

Hence,

$$D = \frac{Q}{2S_F w} = \Delta X(1-\Theta)C - \frac{\Delta X^2}{2K} \qquad [II-63]$$

where S_F is the energy slope and w is the flow width. If the energy slope is approximated by the bed slope, then R can be defined as:

$$\Theta = \frac{1}{2} - \frac{kQ}{(2)wS_0(\Delta X)^2} \qquad [II-64]$$

where S_0 is the bed slope.

The assumption used to develop these relationships for K and R are only valid if a representative discharge and flow width can be determined. Koussis (1978) reports that average values of Q and w could be used.

In general, agricultural stormwater management systems in the flatwoods behave as a series of reservoirs with water levels controlled generally by pumps or weirs. A channel routing method was included as a means of combining hydrographs, and in the event that channel routing is required, a method would be available.

The following example will illustrate how the model operates. See schematic layout, Figure II-10.

I. <u>Given</u>

A. Proposed Acreages

1. Total = 640 ac
2. Reservoir (Detention) = 60 ac
3. Laterals = 40 ac
4. Citrus grove = 540 ac

B. Elevations

1. Laterals extend from elevation 0' (bed) to 4' (banks)
2. Grove extends from elevations 4' to 8'
3. Grove control elevation is 0'
4. Reservoir control elevation is 5' to maintain wetlands

C. Depth to water table = 4.0 ft

D. Pumping rate from grove = 97.6 cfs (4 in./day)

E. Pumping schedule on at 1.5', off at 0'

F. Design storm: 25 year, 3 day = 9.5 in.

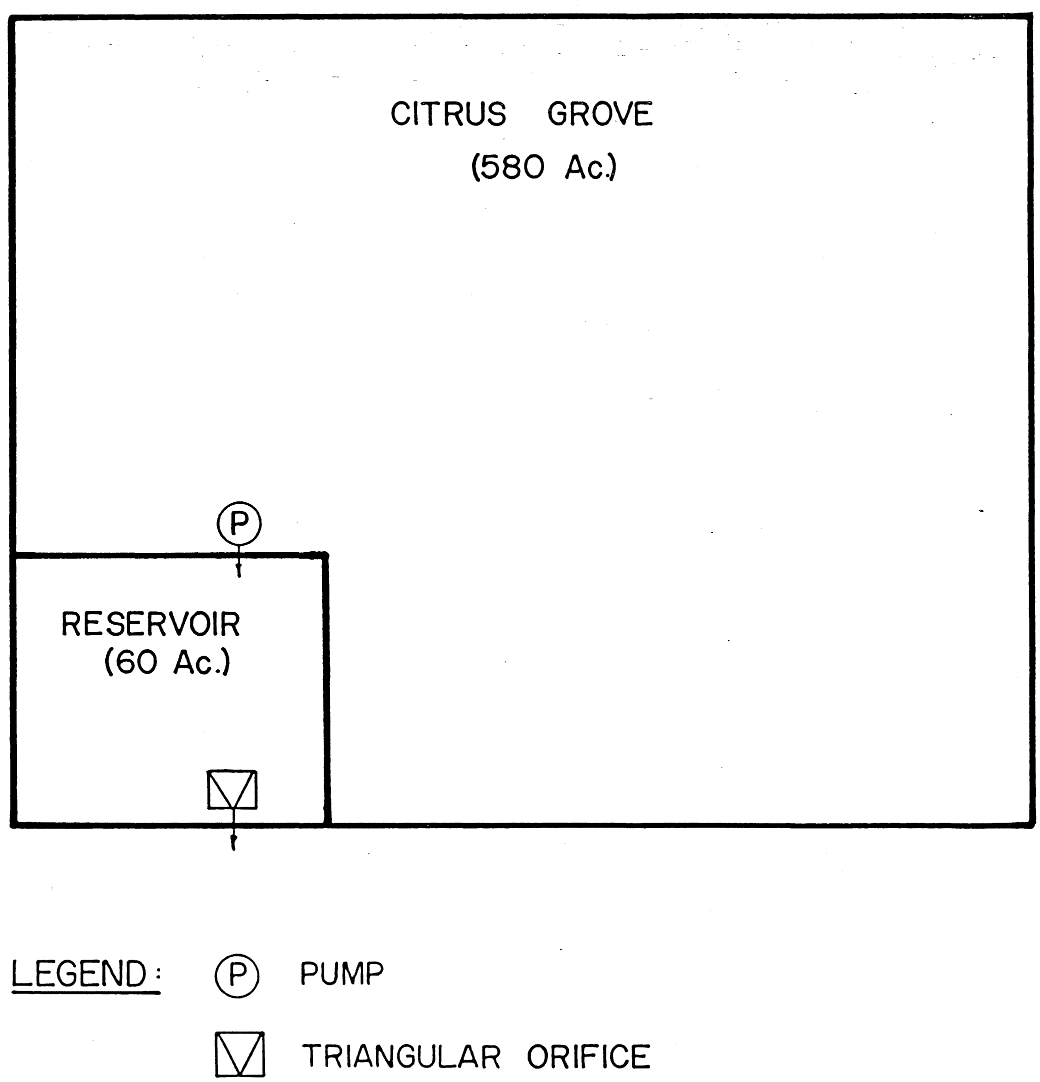
The systems were modeled as shown in the schematic below:

Basin 1 (Grove) → Reservoir #1 (Grove Area Storage) → Channel #3 (Pump Discharge into Detention Reservoir --null structure) → Reservoir #4 (Detention Reservoir)

Basin 2 (Rainfall into Detention Reservoir) → Channel #2 (Rainfall--null structure) → Reservoir #4 (Detention Reservoir)

Reservoir #4 (Detention Reservoir) → Triangular Orifice

FIGURE II - 10: EXAMPLE SITE
(NOT TO SCALE)



II. Computations

A. Grove (580 Acre)

1. Compute pervious/impervious (P/I)

540 Acre grove

40 Acre laterals (assumed impervious)

%I = (40/580)*100% = 7%

%P = 93%

2. Compute soil storage

a. Average depth to water table = 4.0 ft

b. From Heatwole (1986) and "ARS" storage curve the following storages were calculated:

Thickness of Soil Layer (ft)	Depth (ft)	Cumulative* Storage Values (in)	Storage in Each Layer (in)	Pervious** Storage (in)
0.11	0.11	0.075	0.075	0.070
0.56	0.67	0.594	0.519	0.483
0.67	1.33	1.63	1.036	0.965
0.67	2.00	2.92	1.29	1.201
0.67	2.67	4.26	1.34	1.248
0.66	3.33	5.58	1.34	1.248
0.67	4.00	6.92	1.34	1.248
			6.92	6.44

* Storage = $S_{ARS} * (\frac{1}{1 - Ful})$

where Ful = Fraction of total storage held at field capacity

Used Ful = 0.25

** Pervious Storage = Storage in each layer * % Pervious area

3. Runoff Hydrograph

Used unit hydrograph method

From Capace (1984)

$$L = 3.0 + 0.34(A^{0.11})(w+1)^{0.71}$$

For A = 580, w = 0%

L = 3.69 Hours

B. Detention Reservoir

1. Soil storage

The same storage values for the grove were used. In this case the Reservoir is assumed impervious. Therefore, the "storage filled" parameter in the runoff calculation was set to 1.0 to simulate impervious surface.

2. Runoff hydrograph

To simulate rainfall into the detention reservoir, an instantaneous hydrograph is generated.

III. <u>Open Surface Storage</u>

A. Grove--See Table II-2

B. Reservoir--See Table II-2

CITRUS GROVE
(580 AC)

P = 97.6 cfs
(4 in./day)

RESERVOIR
(60 AC)

P Dump

Triangular Orifice

While this example doesn't utilize the full capabilities of CRWT.HYDRO, it does illustrate the model's ability to compute and combine hydrographs from basins with different runoff characteristics and route the resulting hydrograph through a stormwater detention structure in addition to the computations performed by the original CREAMS-WT water quality model.

TABLE II-2

OPEN SURFACE STORAGE

	STAGE (ft)	STORAGE (acre-ft)
GROVE:	0.0	0.0
	1.0	5.0
	2.0	20.0
	3.0	45.0
	4.0	80.0
	5.0	188.0
	6.0	430.0
	7.0	800.0
	8.0	1320.0
RESERVOIR:	0.0	0.0
	1.0	60.0
	2.0	120.0
	3.0	180.0
	4.0	240.0

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APPENDIX II-1
CURVE NUMBER SELECTION PROCEDURE

SCS NEH-4, Chapter 9 (USDA-SCS, 1972b) presents a standard procedure for determining runoff curve numbers. The following outline describes the SCS procedure as adapted for and applied to the flatwoods watersheds of this study.

I. Information and Equipment
   A. SCS soil survey map of watershed.
   B. Soil types and hydrologic classifications (NEH-4 Table 7.1)
   C. Watershed crop cover and soil condition information.
   D. Runoff curve numbers for hydrologic soil-cover complex (NEH-4 Table 9.1).
   E. Curve number adjustment table for AMC (NEH-4 Table 10.1).
   F. A planimeter, digitizer, or grid overlay.

II. Curve Number Estimation Procedure
   A. Document each soil type occurring on the watershed.
   B. Document hydrologic class (or classes) for each soil type from Table Nof SCS NEH-4.
   C. Estimate hydrologic class based upon effectiveness of drainage improvements. For a soil classed as A/D, assign A if very well drained and D if drainage is not sufficient to maintain the water-table well below the surface. Aerial photographs and USGS topographic maps are useful in determining the extent of drainage improvements.
   D. Estimate hydrologic condition as judged from site inspection and Table 26.
   E. Determine land use patterns over watershed from aerial photographs, USGS topographic maps or site inspection.
   F. Determine appropriate curve number for each cover-soil complex (soil class, condition and land use combination) from Table 27 or SCS NEH-4 Table 9.1.
   G. Determine fractional area occupied by each cover-soil complex.
   H. Calculate overall watershed curve number (CN_{II})

APPENDIX II-2

RELATIONSHIP BETWEEN CURVE NUMBER AND AVAILABLE STORAGE

In the SCS runoff equation, the storage parameter, S, is generally determined through a second parameter, the curve number. Curve number is related to S by the function,

$$S = \frac{1000}{CN} - 10$$

The CREAMS version of the curve number method uses this relationship to determine the maximum value of S as,

$$Smx = \frac{1000}{CN_I} - 10$$

where CN_I is the curve number at antecedent moisture condition I (AMC I) which corresponds to dry conditions. An explicit function is used in CREAMS to determine the CN_I value from the input curve number which is for average moisture (AMC II). This function was curve fitted to the corresponding curve numbers for the different antecedent moisture conditions as tabulated by the SCS and is:

$$CN_I = 16.91 + 1.348(CN_{II}) - 0.01379(CN_{II})^2 + 0.0001177(CN_{II})^3$$

For reference, values of the input curve number (AMC II) and the equivalent maximum storage as calculated by the CREAMS algorithms are tabulated here.

Curve number (AMC II) and equivalent maximum storage (Smx) as calculated by CREAMS.

CN	Smx (ins)
100	0.24
99	0.46
98	0.69
97	0.93
96	1.17
95	1.41
94	1.67
93	1.93
92	2.19
91	2.46
90	2.74
89	3.02
88	3.31
87	3.61
86	3.91
85	4.22
84	4.54
83	4.87
82	5.20
81	5.54
80	5.89
79	6.25
78	6.61
77	6.99
76	7.38
75	7.77
74	8.17
73	8.59
72	9.02
71	9.45
70	9.90
69	10.68
68	10.84
67	11.32
66	11.82
65	12.34
60	15.16

Appendix II-3. SFWMD - ALLOWABLE DISCHARGE FORMULAS (SFWMD, 1986)

Canal	Allowable Runoff	Design Frequency
C-1	$Q=\left(\frac{112}{\sqrt{A}}+31\right)A$	10 year
C-2	Essentially unlimited inflow by gravity connections southeast of Sunset Drive; 54 CSM northwest of Sunset Drive	200 year +
C-4	Essentially unlimited inflow by gravity connections east of S. W. 87th Avenue	200 year +
C-6	Essentially unlimited inflow by gravity connections east of FEC Railroad	200 year +
C-7	Essentially unlimited inflow by gravity connection	100 year +
C-8	Essentially unlimited inflow by gravity connection	200 year +
C-9	Essentially unlimited inflow by gravity connection east of Red Road; 20 CSM pumped, unlimited gravity with development limitations west of Red Road or Flamingo Blvd.	100 year +
C-10	-------------------------------	200 year +
C-11	20 CSM west of 13A; 40 CSM east of 13A	---------
C-12	90.6 CSM	25 year
C-13	75.9 CSM	25 year
C-14	69.2 CSM	25 year
C-15	70.0 CSM	25 year
C-16	62.6 CSM	25 year
C-17	62.7 CSM	25 year
C-18	41.6 CSM	25 year
C-19	57.8 CSM	
C-23	31.5 CSM	10 year
C-24	30.25 CSM	10 year
C-25	$Q=\left(\frac{47}{\sqrt{A}}+28\right)A$ (Under review)	10 year
C-38	31.1 CSM (Subject to restrictions of Basin Rule)	10 year
C-40, 41, 41A	35.4 CSM	10 year
Hillsboro Canal (east of S-39)	35 CSM	25 year
North New River (East of S-34)	70.8 CSM	25 year
Everglades Ag. Area (all canals)	20 CSM	5 year
L-28	11.8 CSM	--------
C-51	35 CSM east of Turnpike; 27 CSM west of Turnpike (Subject to restrictions of Basin Rule)	10 year
C-100, 100A, 100B, 100C, 100D:	$Q=\left(\frac{104}{\sqrt{A}}+43\right)A$	10 year
C-102	$Q=\left(\frac{119}{\sqrt{A}}+25\right)A$	10 year
C-103N, C-103S	$Q=\left(\frac{107}{\sqrt{A}}+39\right)A$	10 year
C-110	$Q=\left(\frac{137}{\sqrt{A}}+9\right)A$	10 year
C-111	$Q=\left(\frac{117}{\sqrt{A}}+29\right)A$	10 year
C-113	$Q=\left(\frac{142}{\sqrt{A}}+3\right)A$	10 year

Definitions:
Q = Allowable runoff in cfs (cubic feet per second) A = Drainage area in square miles CSM = cfs per square mile

PART III NUTRIENT ATTENUATION IN WATER MANAGEMENT SYSTEMS

Introduction

Stormwater management, which is essential for successful intensive agricultural production in high-water-table soils common to South Florida, is now required to meet both discharge rate and quality criteria when discharged to waters of the state. Section II of this report addressed design of the facilities required to meet discharge rate criteria. This Section will address nutrient attenuation in the impoundment facilities of a water management system.

Objectives

The purpose of this study was to review research findings and computer modeling methods applicable to Florida's flat, high-water-table soils and to develop modeling procedures to account for nutrient attenuation in water management impoundments for control of nitrogen and phosphorus. Specific objectives were:

A) Identify the processes that transform nitrogen and phosphorus from soluble, mobile forms for temporary or near-permanent immobility and removal from drainage water.

B) Develop a computer model capable of analyzing nutrient attenuation in wetland and manmade impoundments under different storage and flow-through regimes. This model is to be compatible, but not necessarily linked, to the model to be developed as described in Section II.

Nitrogen Processes

General:

Within a wetland, flooded field, or detention/retention pond different physical, chemical and biological processes affecting nitrogen occur. The biological transformations include: immobilization of inorganic nitrogen by microorganisms and plants to form organic compounds, ammonification - the decomposition of organic nitrogen to NH_4, nitrification - microbial oxidation of NH_4^+ to NO_2^- and NO_3^-, denitrification - the reduction of NO_3 or NO_2 to N_2O and N_2, and nitrogen fixation - the reduction of N_2 to NH_3. Chemical reactions taking place in the soil nitrogen cycle include: ammonia volatilization or sorption - the release or uptake of atmospheric NH_3 by plants or soils, exchange of NH_4 from soil cation exchange sites to soil solution, ammonium fixation - the entrapment of NH_4 within interlayers of clay minerals, and chemical denitrification caused by acid pH or elevated temperatures. The predominant process concerning nitrogen in the NH_3 form is plant assimilation. The predominant process concerning nitrogen in the NO_3^- form is denitrification which accounts for 95 percent of the nitrate lost (Heatwole, 1986). Some of these processes will be looked at later.

Denitrification:

Denitrification is the process of microbially reducing nitrate to products in gaseous forms (primarily dinitrogen and nitrous oxide). Nitrate is converted to nitrite by nitrobactor bacteria and the nitrite is subsequently reduced to nitrogen gas or nitrous oxide. Denitrification occurs under at least partially anaerobic conditions with rates increasing with increasing temperature. Experimental results show slow rates at about 3 degrees celsius increasing rapidly to about 35 degrees celsius, increasing slower from 35-60 degrees celsius, and finally dropping off at temperatures greater than 60 degrees celsius (Krottje, 1980). Denitrification is also a function of the amount of organic matter and the pH. High organic matter content yields rapid denitrification, and an increasing pH causes the denitrification rate to decrease (Krottje, 1980). Denitrification requires adequate residence time for nitrate removal. This residence time is on the order of 12 - 24 days, but is highly variable (Heatwole, 1986). In a controlled situation, the water level may be manipulated for optimum nitrogen gas escape (Good, et al.,1978). The majority of the denitrification happens in the soils instead of in their overlying waters. Adjusting the depth of water in the system can optimize the amount of N_2 gas (formed by denitrification) which can escape. The quantity of dissolved oxygen is reduced by the existence of a dense cover of floating aquatics. This leads to anaerobic conditions within the treatment system, and denitrification will be favorable under these conditions. This leads to an increase in NO_3^- removal (Reddy, 1983).

Krottje (1980) found he could fit first order kinetics to the denitrification rate with rate constants varying from 0.040/day to 0.192/day. These convert into nitrate losses from 600 to 2900 g N/ha-day (Krottje, 1980). These constants were evaluated experimentally for 14 Florida soils. The rate constants are a function of organic carbon and pH, expressed as:

$$K_1 = [(8.9 \times 10^{-4})(ocw) - (3.9 \times 10^{-4})(ocw)(pH_{6.5}) + 0.002]^{0.5} \qquad \text{(III-1)}$$

where:

K = the rate constant

ocw = the organic carbon content by weight percent

$pH_{6.5}$ = deficit pH below 6.5

NH_3 Volatilization:

Ammonia volatilization is the release of NH_3(gas). In this way, plants, soil, and water can give off NH_3 thus removing nitrogen from the system into the atmosphere.

Nitrous Oxide Evolution:

When nitrous oxide enters the atmosphere, it has the hazardous effect of breaking down the ozone layer.

Krottje found a significant correlation between nitrous oxide evolution and the reciprocal of the denitrification rate, and he also found that nitrous oxide gas evolution accounted for about 0.2-6.5% of the nitrate consumed. This corresponds to 5-47 g N/ha-day (Krottje, 1980).

Phosphorus Processes

General:

Concerning eutrophication in South Florida, phosphorus is the nutrient of primary concern because it is a limiting factor in the process, and it is the most difficult nutrient to control. Phosphorus exists in both particulate and dissolved forms. Particulate phosphorus forms include adsorbed, organic, precipitates, and minerals. Dissolved forms include orthophosphate, organic polyphosphate, and other organic phosphorus compounds. Organic phosphorus compounds undergo microbial decomposition (mineralization) to become dissolved phosphates, and dissolved phosphates undergo microbial synthesis (immobilization) to form organic phosphorus compounds. In most soils, little phosphorus is lost by leaching due to the soil's strong ability to hold phosphate. In these soils phosphorus is transported by being held to eroded sediments carried by runoff. This is not the case in Florida's sandy soils. Here phosphorus leaches through the soil because of the phosphate's low affinity for the soil. In this case phosphorus is mainly carried in solution into surface and groundwaters. Besides the sandy nature (low clay content) of Florida soils, another factor causing increased phosphorus leaching is the acidity of the soil. Phosphorus leaching increases with decreasing pH (Heatwole, 1986). There are two primary mechanisms for removing phosphorus from waters treated in wetlands or some other type of detention/retention reservoir. Those mechanisms are 1) precipitation or sorption of phosphorus onto organic matter, and 2) assimilation (uptake) by algae and macrophytes.

Desorption:

Adsorption-desorption has been found to be an irreversible process with the adsorption rate being greater than the desorption rate (Enfield and Ellis, 1983). Barrow (1980) found the rate of phosphorus desorption to be decreasing proportional to the cube root of the elapsed time.

Adsorption-Precipitation:

Even through extended study into the chemistry of phosphorus, it is still impossible to accurately separate the relative influences of the processes of adsorption and precipitation and describe their interactions (Sanchez and Uehara, 1980). When phosphorus concentrations are high (molar range), precipitation occurs, and when phosphorus concentrations are low (milli-molar range) adsorption occurs (Sample et al., 1980). Differentiating between the two processes is not of major importance as long as they can be accurately lumped and modeled together as one process. The rate of adsorption-precipitation is a function of the redox potential, the soil pH, and the availability of reactive compounds (Heatwole, 1986).

UPTAKE BY PLANTS

General:

Different types of aquatic plants assimilate nutrients in different ways. Rooted submersed plants take up nitrogen from both soil and water. Most emergent plants take up most or all of their nitrogen from the soil (Greeson et al., 1979). Based on this fact, emergent plants have little direct impact on removing nitrogen from passing waters. Epiphytes remove their nitrogen directly from the water (Greeson et al., 1979) therefore having a pronounced effect on the nitrogen concentration of passing waters. Emergent and submersed plants remove phosphorus from the water and underlying sediments (Greeson et al., 1979). A portion of the phosphorus is translocated in fall to the plants' underground structures while the remainder in the plant-epiphyte portion becomes litter and returns to the water.

Experimental Results:

Studies by DeBusk et al., 1983, on nitrogen and phosphorus removal from wastewater in a water hyacinth pond show actively growing hyacinths with uptake rates of nitrogen and phosphorus of 0.6 g N/m^2-day and 0.2 g P/m^2-day respectively. Effects of harvesting are shown in total removal rates. For nitrogen, harvested ponds removed 362 mg N/m^2-day while non-harvested ponds removed 55 mg N/m^2-day. For phosphorus, harvested and non-harvested values were 115 mg P/m^2-day and 15 mg P/m^2-day, respectively (DeBusk et al., 1983). In the non-harvested systems crop density increased to 35-40 Kg wet wt./m^2. At these density levels and above, we see a reduction in net productivity (Taylor and Stewart, 1978) and a decline in the nutrient uptake rates (Boyd, 1976). This study also shows overall average growth rates of 15-25 g dry wt./m^2-day and a complete plant coverage, harvested pond growth rate of 16 g dry wt./m^2-day (DeBusk et al., 1983). In a 1983 study by Reddy the nutrient loss rate was modeled by a first order equation. Rate constants for nitrogen removal using pennywort, cattail-elodia, water hyacinth, and a blank were 0.188, 0.184, 0.039, and 0.025/day respectively. Constants for phosphorus removal were 0.025, 0.024, and 0.028/day and no blank was run (Reddy, 1983).

Wetland Studies

Numerous studies have been done to evaluate the worth of wetlands in treating wastewater and runoff, whether it be municipal, industrial, or agricultural. The studies of primary concern in this research are those dealing with the removal of nitrogen and phosphorus from agricultural runoff.

In a study on the use of freshwater wetlands for treating wastewater in Central Florida, results showed a total phosphorus reduction from 6.4 mg/l to 0.12 mg/l (98% reduction) and a total nitrogen reduction from 15.3 mg/l to 1.6 mg/l (89.5% reduction) (Kadlec and Tilton, 1979).

Studies by the South Florida Water Management District on the efficiency of detention/retention sites in nutrient removal showed that a marsh/pond system removed 62% of the total nitrogen and 58-88% of the total phosphorus. The studies also showed that a meadow/marsh/pond system removed 79% of the total nitrogen and 75-92% of the total phosphorus (Goldstein, 1983). Chandler Slough is a 1000 acre marsh in the Lower Kissimmee River Basin which is influenced by the backwaters of canal C-38. A two year study on the marsh's efficiency of removing phosphorus from watershed runoff showed net phosphorus removals of 6.7% (5.1 lb/acre) in 1975 and 34.8% (17.2 lb/acre) in 1976 (Ammon et al. 1981). This study also showed that the first heavy rainfall event (flush) of each wet season resulted in the loss of a great deal of the phosphorus deposited in the marsh the preceding year.

Another study using marshes for treating wastewater showed promising results as reduction rates were very good. The system reduced input loads of 38.03 g P/m^2 to a load of 0.94 g P/m^2 at the outlet (Dolan et al., 1981). Although the marsh was extremely successful in reducing phosphorus output in the first year it was implemented, the investigators proposed that the long-range success of the marsh at removing phosphorus would rely on factors such as peat productivity and the capacity of the marsh soil for adsorbing phosphorus.

In a slightly different, but related situation, McPherson et al., 1976, studying nitrogen and phosphorus uptake in canals in the Everglades Conservation areas, found a two percent per mile reduction in phosphorus and a four percent per mile reduction in nitrogen.

Upland Detention/Retention Project

One of the more extensive studies on the use of wetlands in treating non-point source pollution was the South Florida Water Management District's Upland Detention/Retention Demonstration Project which spanned over three years. One of the project's goals was to evaluate and compare two types of wetlands on their ability to act as low cost/low energy pollution sinks (South Florida Water Management District, 1986).

Ash Slough in Okeechobee County is an 8.1 hectare, naturally low-lying depression. It holds water on a periodic basis, having periods of both flooding and drought. Armstrong Slough is found in Osceola County and is a 12.1 hectare flow- through wetland (marsh). This area remained flooded to different extents constantly throughout the study.

Field measurements included those for flow, total nitrogen, total phosphorus, inorganic nitrogen, and orthophosphate. Results for Ash Slough are given on the basis of five separate inflow/outflow events separated by the drying out of the wetland. Results for Armstrong Slough are given in yearly budgets over the three year study period since the wetland held water at all times. For results see Tables III-1a, b.

Armstrong Slough was found to have a net uptake effect for nitrogen and phosphorus only in the first two years of the study. In the last year of the study it was a net exporter. In that year it was also a net exporter of water, and it only reduced dissolved inorganic nitrogen and orthophosphate. This again shows how site and event specific these types of evaluations can be.

In this study it is important to note that the majority of the nutrient uptake is passive uptake. Passive uptake is that due to storage or reduction in flow. Above that amount is active uptake, which occurs because of some adsorption, assimilation, or alteration.

% Active Uptake = % Nutrient Uptake - % Flow Reduction

ASH SLOUGH

	Percent Reductions				
	Flow	Inorganic N	Total N	Ortho P	Total P
Event 1	30.0	78.4	29.0	39.3	41.0
Event 2	82.8	92.5	85.1	82.7	82.8
Event 3	25.8	84.0	13.7	38.7	36.9
Event 4	24.1	34.4	6.3	30.5	28.6
Event 5	24.5	20.9	1.5	57.1	42.6

Table III-1a – Percent reductions for Ash Slough based on separate events. (SFWMD, 1986).

ARMSTRONG SLOUGH

	Percent Reductions				
	Flow	Inorganic N	Total N	Ortho P	Total P
Year 1	58.6	86.0	65.2	83.3	79.2
Year 2	29.9	65.6	32.0	41.7	43.0
Year 3	-9.4	50.8	-5.2	3.6	-25.1

Table III-1b – Percent reductions for Armstrong Slough based on yearly budgets. (SFWMD, 1986).

Management Practices

Possible Techniques:

There are a number of management practices that can be used to help optimize conditions for nutrient assimilation. Improved farming and ranching practices could, of course, help cut down nutrient loads, but we will only consider detention/retention practices in site management. A major problem is flushing at undesirable times. This can be avoided by not allowing nutrients to enter surface waters during summer aquatic weed and algal blooms (Good, 1978). This can also be viewed as a land site practice. Another management practice is to regulate outflow from detention structures. By diking or regulating outflow from detention ponds and wetlands, prevention of nutrient washout to receiving waters at objectionable times can be attained. Flushing at opportune times is another beneficial practice. Detain the water in the detention site until late fall when nutrient mobilization occurs. At this time, flush the site and divert or pump the washed out material to fields for overland flow application. A final practice to be considered is harvesting. Since this technique is probably the most promising, it will be treated in more detail in another section. Howell et al., 1981, also suggest dredging sediments and/or releasing the resolubilized phosphorus stored in the sediment to receiving waters when its potential for environmental impact is lowest.

Harvesting:

Harvesting is a practice which allows the ultimate removal of nutrients from the wetland or detention pond. Harvesting young tissue forces nutrient translocation and cuts secretion losses to a minimum. For this reason, harvesting should be done frequently during the growing season so that young vegetation tissue is being cut (Good, 1978). DeBusk et al., 1983, found that in water hyacinth, not harvesting allowed crop density to increase to 35 to 40 kg wet wt./m^2, which may cause net productivity and nutrient uptake to decline. Studies made in Coral Springs, Florida on water hyacinth based treatment plants showed significant differences in nitrogen and phosphorus reduction depending upon whether vegetation was harvested or not harvested. Phosphorus reduction was 115 mg/m^2-day for harvested systems as opposed to 15 mg/m^2-day for non-harvested, and nitrogen reductions were 362 mg/m^2-day and 55 mg/m^2-day for harvested and non-harvested respectively (DeBusk et al., 1983).

Effects of Hydraulic and Nutrient Loading

Goldstein, 1983, found that as long as the total nitrogen input load was less than 30 ppm, a marsh/pond system would capture 62 percent of the total nitrogen, and a meadow/marsh/pond system would capture 89 percent of the total nitrogen. Goldstein also found that phosphorus inputs could not be greater than 11 mg/l for substantial phosphorus reduction. The types of vegetation present and the efficiency of nutrient removal are also strongly dependent upon the depth of the water in the system. Kadlec and Tilton, 1979, looked at the effects of hydraulic loading on nutrient removal and found that rates of greater than one inch per week hurt removal efficiency.

Studies investigating the effects of flushing rates on detention of phosphorus in an aquatic system show that phosphorus loading is directly proportional to flushing rate, phosphorus loading is inversely proportional to detention coefficient, and detention coefficient is inversely proportional to flushing rate, where the detention coefficient is the fraction of the phosphorus input not lost in the outflow, the flushing rate in detention exchanges/month, is the monthly discharge divided by the volume, and the phosphorus loading is the total phosphorus input divided by the reservoir area (Kaul et al., 1984). If phosphorus loading is in g/m^2-month, flushing rate in detention exchanges/month, and detention coefficient unitless, straight line relationships can be applied to the study results. Phosphorus loading versus flushing rate has the equation:

$$y = 0.28 + 0.23x \qquad \text{(III-2)}$$

with a correlation coefficient of 0.79, phosphorus loading versus detention coefficient has the equation:

$$y = 1.583 - 1.98x \qquad \text{(III-3)}$$

with a correlation coefficient of -1.481, and detention coefficient versus flushing rate has the equation:

$$y = 0.79 - 0.21x \qquad \text{(III-4)}$$

with a correlation coefficient of 0.97.

Sediment-Water Interchange

Sediment-water interchange is very important in nitrogen and phosphorus removal from systems such as wetlands, flooded fields, and detention/retention reservoirs, especially those with limited aquatics or non-harvested vegetation where nutrient removal happens mainly through the biological, chemical, and physical processes occuring in the water and sediments. Some of these processes such as nitrification/denitrification, NH_3 volatilization, precipitation, and adsorption/desorption have been previously discussed. Studies into the effects of temperature, water turbulence, and sunlight on the exchange rates of nitrogen and phosphorus showed that neither seasonal temperature variations, turbulent versus quiescent water, or subjection to daylight or darkness had much effect on the rates (Reddy,1983b). In explanation of his results, Reddy accounts for the rapid loss of NH_4+ through NH_3 volatilization and nitrification. The nitrification of NH_4+ to NO_3- caused increased levels of NO_3- in the water. NO_3- removal is dependent on denitrification and uptake by algae and macrophytes. Reddy correlated high organic matter content with a high potential for reduction in nitrate levels. This follows our previous observation that a high organic matter content increased the denitrification rate (Krottje, 1980). Reddy, 1983b, found sediment-water exchange rates of total phosphorus for reservoirs and flooded organic soils. The differences between the two systems are mainly the depth and density of the soil.

The reservoir has a shallower, more dense soil (hard packed bottom) while the water column depth of both systems is the same. Following the convention that (-) means phosphorus disappeared from the water, the exchange rates for the reservoir were -1.04 to -0.60 mg P/m^2-day and for the flooded soil were -0.46 to 0.35 mg P/m^2-day (Reddy, 1983b).

Wetlands, Detention/Retention Criteria

Department of Environmental Regulation:

Detention/retention systems must provide the capacity to hold the runoff from the first inch of rainfall, except for drainage areas of less than 100 acres which may alternatively provide for the first half inch of runoff. Retention and detention basins shall provide the capacity for the designated runoff volume within 72 hours following the storm event. Basins should be fenced off or in some other way be made inaccessible to the public unless their side slopes are less than (milder) 4:1 to a depth of two feet below the control elevation. Wetlands available for use in stormwater management must be connected to other waters by artificial watercourses or solely by an intermittant watercourse.

South Florida Water Management District:

Wet detention must provide for the first inch of runoff or the total runoff of 2.5 inches times the percentage of the project area that is impervious, whichever is greater. Dry detention and retention should be 75 and 50 percent of this value respectively. Discharge from projects should meet the state water quality standards found in Chapter 17-3 of the Florida Administrative Code. Viable wetlands and their buffer areas and natural systems composed of distinct, interdependent upland/wetland systems shall be preserved. Natural wetlands may be replaced by man made wetlands of equivalent productivity, and small isolated wetlands may be "traded off" for larger upland/wetland systems of equivalent productivity.

Structural facilities must be provided for all design discharges. No discharge will be made below the control elevation except through emergency devices which must have locking devices whose keys are held by the District or another acceptable governmental agency. Bleed-down through gravitational devices shall account for one-half inch of the detention volume to be discharged in the first day, and pumping devices shall provide 20 percent of the detention volume in a day. Gravity devices shall be V-notch or circular in cross-section so as to increase detention during minor events. The device shall have a cross-section of at least six in^2, a minimum dimension of 2 inches, and a minimum angle of 20 degrees for V-notches. For wet detention/retention the minimum area is one half an acre, and the minimum width for linear areas longer than 200 feet is 100 feet. Irregular shapes may be narrower than 100 feet in some places but should average at least 100 feet.

Modeling Nutrient Attenuation in Water Management Impoundments

In order to analyze nutrient capture or loss in various types of impoundments in a water management system, it is necessary to trace the movement of water through the impoundment, and to account for the processes that affect nutrients in the water during each increment of residence time in the impoundment. A computer model was developed for this analysis. It is compatible with the model developed by Burleson, 1987, as described in Section II of this report. In describing the nutrient attenuation model, as follows, considerable detail as to various rate coefficients is presented. These are based on the best information determined in literature review during this study. If these coefficients are improved through further research, they can be changed in the model files.

Model Inputs:

The input data set for the model consists of runoff data, nutrient data, vegetation data, and assorted other parameters. This data is read from three separate files.

One file contains just the runoff data. This data is received from CREAMS WT by way of a hydrograph and stream routing procedure developed by Burleson, 1987. The values are read at evenly incremented time steps as the inflow rates into the first cell. These values are read in continuously until inflow (runoff) ceases.

A second file contains all of the data concerning nitrogen and phosphorus concentrations and vegetation densities within the pond. In nested loops passing through the cell indices, the initial concentrations of nitrogen and phosphorus and the initial plant densities within each cell in the pond are read. Then, as with the volume inflow rates, the inflow concentrations are read at each time step. As will be shown later, this occurs on a much longer time step than it does for the routing data. This is because of the slower dynamics of these processes and the fact that most models only give daily nutrient loads. Therefore, we will input nutrient concentrations on a daily basis, or make some assumptions as to load variability and read values on an hourly or multiple hour basis.

The third file contains all the other parameters necessary for the model to run. It contains routing parameters such as the cell indices, the cell width (ft), cell area (ft^2), routing time step (min), total simulation time (days), and initial pond height (ft). Also concerning the flow routing are parameters describing the outfall. These include the cell at which the outlet exists and the type of outfall. The possible outlet structures are a free weir discharge, a circular or triangular orifice, a V-notch, or a combined weir and V- notch. Depending on the structure chosen, some of the following parameters must be known: the weir crest elevation (ft), the weir length (ft), the circle diameter (ft), the circle bottom elevation (ft), the V-notch or triangle's angle (degrees), the V-notch or triangle's height (ft), and the V-notch or triangle's vertex elevation (ft). Parameters dealing with the nutrient and plant growth processes are also located in this file. These include: the chosen time step (hrs.), the denitrification first-order rate constant, the exponential growth rate constant, constants for the precipitation- adsorption equation, constants the desorption equation, the number of times to harvest during simulation, and the percentage of vegetation to harvest each time.

Note: All elevations are referenced to the pond bottom which is chosen as the datum.

Initial and Boundary Conditions:

Setting the initial conditions for the model consists of specifying the initial pond height, the initial nitrogen and phosphorus concentrations in each cell, and the initial vegetation density in each cell. The boundary conditions are used to set the flows across the pond boundaries, with exception to the inlet and outlet, equal to zero. That is, there is no flow into or out of the pond except at the specified points.

Growth:

The growth model used in the program is based on findings of studies by DeBusk et al., 1983, using water hyacinths. From these findings the overall growth rate ranges from 15-25 g dry wt./m^2-day depending upon plant density. Plant growth rate was modeled to reach an equilibrium growth rate once the vegetation density reaches 34 kg wet wt./m^2. Therefore, the growth rate in g dry wt./m^2-day is modeled as an exponential function of plant weight (density) in kg wet wt./m^2 with a maximum value of 25 g dry wt./m^2-day for new plants and a minimum value of approximately 15 g dry wt./m^2-day for plant weight above 35 kg wet wt./m^2. The equation is:

$$\text{growth} = 10\ e^{-k(\text{plant wt.})} + 15 \qquad \text{(III-5)}$$

and suggested rate constants are in the range of 0.10-0.30. Rate constants could be calibrated using site specific data.

Nutrient Uptake:

The model bases its nitrogen and phosphorus uptake rate equations on data found in research by Ogwada, 1983, on growth and nutrient uptake. The specific data used was for water hyacinth. The data correlates growth rate (g dry wt./m^2-day) to nitrogen and phosphorus uptake rates (mg/m^2-day). Ogwada also correlates the pond concentration of the nutrients to the other parameters, but the first relationship will be used in establishing the model's equations. Knowing the growth rate over a certain period via our forementioned growth model, the uptake rates can be found as a function of growth rate. Plotting the uptake of nitrogen and phosphorus versus growth rate and fitting straight lines yields good results (r^2 values for nitrogen and phosphorus are 0.93 and 0.92 respectively). The equations of the resultant lines are:

$$\text{nitrogen uptake} = 1.57 + 10.18 * \text{growth rate} \qquad \text{(III-6)}$$

$$\text{phosphorus uptake} = 0.20 + 1.02 * \text{growth rate} \qquad \text{(III-7)}$$

where uptake rates are in mg/m^2-day and growth rates g dry wt./m^2-day. Data are shown in Table III-2. Ogwada's uptake rate values for water hyacinth are substantiated by findings of Reddy, 1983. For comparisons, see Table III-3. Note that Reddy's values fall between Ogwada's values for nutrient limiting and nutrient enriched situations for both nitrogen and phosphorus.

Month	Growth Rates g dry wt /m² dy	Uptake Rates mg/m² dy		Pond Conc. µg/ml	
		N	P	N	P
Jan	-2.0	-25.0	-2.0	3.31	0.71
Feb	3.0	40.0	4.0	0.23	0.38
March	3.0	30.0	3.0	0.33	0.20
April	7.5	75.0	8.0	0.03	0.11
May	11.0	110.0	14.0	0.16	0.16
June	11.0	95.0	9.0	0.06	0.10
July	7.5	75.0	6.0	0.23	0.17
Aug	8.5	120.0	10.0	0.13	0.18
Sept	-	-	-	0.04	0.41
Oct	-	-	-	0.35	0.18
Nov	2.0	15.0	2.0	0.08	0.18
Dec	0.0	5.0	0.5	0.26	0.14

Table III-2 – Pond concentrations, growth rates, and uptake rates for waterhyacinth. Ogwada, 1983.

	Reddy's given rates(g/m² dy)		Reddy's converted rates(kg/ha yr)		Ogwada's rates (kg/ha yr)			
	N	P	N	P	N		P	
					limit	enrich	limit	enrich
Pennywort	0.190	0.050	693.50	182.50	—	—	—	—
Water Hyacinth	0.20	0.024	730.0	87.60	169.0	1460	20.1	340.0
Cattail-Elodia	0.12	0.004	438.0	14.6	—	—	—	—
Cattail	0.05	0.001	182.5	3.65	117	334	9.2	83
Elodia	0.07	0.003	255.5	10.95	—	—	—	—
Algae	0.02	0.003	73.0	10.95	—	—	—	—

Table III-3 – Comparison of nutrient uptake rates for waterhyacinth in studies by Ogwada, 1983, and Reddy, 1983.

Harvest:

Without regular and frequent vegetation harvest, neither optimal growth and uptake of nutrients, nor ultimate removal of nutrients from the wastewater can be attained (DeBusk et al., 1983; Goldstein, 1983). For this reason the model contains a simple harvesting subroutine. Harvest is optional, and it is done at uniform times within the total simulation duration. The harvest is also uniform on a cell-wise basis, i.e. the same percentage of density is removed from each cell. The number of harvests and the percentage of density to be harvested are the only inputs. By entering data on a cell- wise basis, requiring a more extensive input data set, spatially non-uniform harvesting could easily be incorporated into the model. Also, time variations in harvesting could be accomplished quite easily.

Denitrification:

The model treats denitrification by a first order kinetics equation as found by Krottje, 1980. Krottje also found that the rate constants for this process varied from 0.040 to 0.192/day^{-1} for 14 typical Florida soils. Along with these observed values, a prediction equation was also used to arrive at rate constants based on organic carbon content by weight percentage and pH deficit below 6.5. The equation is:

$$K_1 = [(8.9 \times 10^{-4})(ocw)-(3.9 \times 10^{-4})(ocw)(pH_{6.5}) + 0.002]^{0.5} \qquad \text{(III-8)}$$

For our purposes, we will use observed values. The observed and predicted values can be seen in Table III-4. The first order equation is:

$$CT = CO/e^{kt} \qquad \text{(III-9)}$$

where CT is the concentration at time t, CO is the initial concentration, and k is the rate constant. For convenience in calculating the nutrient concentrations at the end of each time step, we will rearrange the equation to solve for the amount of nitrogen lost due to denitrification. The equation becomes:

$$CL = CO * (1- e^{-kt}) \qquad \text{(III-10)}$$

where CL is the amount of nitrogen lost and CO-CL=CT. Denitrification is calculated on a cell-wise basis.

Soil Type	Rate Constants	
	observed	predicted
Everglades muck	0.192	0.183
Floridian fine sand	0.149	0.115
Astor sand	0.108	0.093
Surrency sand	0.105	0.113
Samsula muck	0.102	0.098
Pickney fine sand	0.087	0.108
Riviera fine sand	0.076	0.050
Chastain silt loam	0.067	0.069
Brighton peat	0.066	0.057
Iberia silt clay	0.063	0.070
Chobee fine silt loam	0.062	0.093
Delray fine sand	0.061	0.088
Eureka fine sand loam	0.049	0.069
Valkaria fine sand	0.040	0.049

Table III-4 – Observed and predicted rate constants used to model denitrification in Florida soils as a first-order equation. Krottje, 1980.

Desorption:

A number of possible empirical equations were available for calculating the desorption of phosphorus. The equation chosen was one developed by Chien and Clayton using a modified Elovich equation (Chien and Clayton, 1980). The equation is :

$$q = (1/b)\ln(ab) + (1/b)\ln(t) \qquad \text{(III-11)}$$

where q is the amount of desorbed phosphorus at time t, and a and b are empirical constants. We begin with the Elovich equation which is:

$$dq/dt = a\ \exp(-bq) \qquad \text{(III-12)}$$

As q approaches 0, dq/dt approaches a, therefore we will regard a as the initial rate. If we assume $q = 0$ at $t = 0$ and integrate the equation we obtain the equation:

$$q = (1/b)\ln(1+abt) \qquad \text{(III-13)}$$

Now an assumption is made which we will check later. If we assume abt is much greater than 1 the equation becomes:

$$q = (1/b)\ln(ab) + (1/b)\ln(t) \qquad \text{(III-14)}$$

Plotting q versus t should be linear with a slope of 1/b and an intercept of (1/b)ln(ab). Constants a and b can be calculated using experiments for given soils and conditions. From the calculated values (see Table III-5) we see that the product of a and b is much greater than 1, therefore our assumption was valid. Studies show that for dissolution of phosphate rock, a is a function of soil type but not a function of the rock source, and b is a function of both the soil type and the rock source (Chien et al., 1980). For phosphate sorption, both a and b are functions of soil type. For phosphate release, b varies widely among soils while a changes very little.

Modeling phosphate release and sorption by the Elovich equation was compared to modeling by parabolic diffusion law, two-constant rate, and second-order reaction equations, and the Elovich equation gave better correlation in every instance. The r^2 values from regression analysis of the Elovich equation results ranged from 0.984 to 0.998.

Soil Type	a	b
Waukegan silt loam (release)	$3.6*10^2$ ppm P/hr	0.089/ppm P
Fargo clay (release)	$2.4*10^2$ "	0.156 "
Langdon loam (release)	$2.9*10^2$ "	0.405 "
Okaihau (sorption)	$1.2*10^6$ μmol P dm^{-3}/hr	0.009/μmol P dm^{-1}
Porirua (sorption)	$1.1*10^4$ "	0.034 "

Table III-5 - Experimentally calculated constants used in the Modified Elovich equation to model phosphorus desorption. Chien and Clayton, 1980.

The desorption of phosphorus could be modeled by another empirical equation simply by exchanging equations in the subroutine and inputting the needed constants and parameters. The desorption of phosphorus is calculated by the model on a cell-wise basis.

Precipitation-Adsorption:

A number of possibilities also existed for modeling the precipitation and adsorption of phosphorus. Of the many empirical equations available, the two most common are the Langmuir and Freundlich equations. Of these two the Freundlich equation was chosen for its simplistic form. The Freundlich equation is:

$$S = KC^n \tag{III-15}$$

where S is the adsorbed phosphate at concentration C, C is the solution concentration of phosphorus, and K and n are empirical constants that must be fitted. K and n are both functions of the native soil.

If a particular situation dictates that the Langmuir equation, a modification of the Langmuir or Freundlich equation, or a completely different empirical equation would be better suited to the problem, the subroutine is set up so that the equations can be easily substituted. Like many of the other processes, precipitation and adsorption are calculated on a cell-wise basis.

Routing:

Inlet flow rates to the pond are received from a stream routing and hydrograph routine developed by Burleson, 1987. The flow is then routed through the pond using an energy balance with an energy dissipation factor. Boundaries across which there are no flow condition are also established, as is the outflow condition at the appropriate cell. The energy balance is solved in both the x and y directions for each cell.

Outflow Options:

The model makes available an outflow option at the appropriate cell. A choice between five available outflow structures is available. The five options are: free weir discharge, V-notch flow, orifice flow (circular or triangular), or combined V-notch and weir flow.

The free weir discharge is calculated by the equation:

$$Q = CLH^{1.5} \tag{III-16}$$

where Q is the discharge in cubic feet per second, L is the weir length in feet, H is the head in feet, and C is a weir coefficient. C is most often taken to be equal to 3.13 making the equation:

$$Q = 3.13\ LH^{1.5} \tag{III-17}$$

The flow through a V-notch outlet is calculated by the equation:

$$Q = 2.5\ TAN(theta/2)\ H^{2.5} \tag{III-18}$$

where theta is the angle of the "V" in degrees, and H is the head on the notch vertex in feet. For a given detention volume the V-notch can be sized using the equation:

$$theta = 2\ TAN^{-1}(0.492\ V_{det}/H^{2.5}) \tag{III-19}$$

where V_{det} is one-half inch of detention volume in acre-feet, and H is the vertical distance from the weir crest to the angle vertex in feet.

The general equation for flow through an orifice is:

$$Q = 4.8AH^{0.5} \tag{III-20}$$

where A is the area of the notch in square feet, and H is the head above the notch centroid. Solving this equation for a circular cross-section gives the equation:

$$Q = 4.8\ (PI*D^2/4)\ H^{0.5} \tag{III-21}$$

where D is the circle diameter in feet. The orifice equation could also be used for a triangular cross-section. In this case the orifice equation becomes:

$$Q = 4.8\ (H_2{}^2\ TAN(theta/2)(H_1 + H_2/3)^{0.5} \tag{III-22}$$

where H_1 is the head above the triangle top, and H_2 is the height from the triangle's vertex to its top. The final option is a combined V-notch and weir flow. Adding these two together, the equation becomes:

$$Q = 3.13LH_1^{1.5} + 4.8(H_2^2\ TAN(theta/2)(H_1 + H_2/3)^{0.5} \qquad \text{(III-23)}$$

where the variables are as defined before. All of these options and their corresponding equations are recommended by the South Florida Water Management District (SFWMD, 1986). The SFWMD also makes some regulations concerning structure sizes and bleed down rates. For orifice structures, the cross-sectional area shall be greater than or equal to six square inches, and the minimum dimension shall be greater than or equal to two inches. For V-notch structures, the minimum angle of the "V" shall be 20 degrees. For gravity bleed down structures such as these, regulations require discharge of one-half of an inch of the detention volume in the first day. The rationale behind the circular or V-notch orifice is that the small cross-section at the bottom allows longer detention times for small events.

Residence Time:

Processes concerning nutrient removal are directly dependent upon a parcel of water's residence time within a treatment area. The residence time of a parcel of water leaving during the present time step is calculated by the equation:

$$RTIME(i) = V(i)/((I(i) + O(i))/2) \qquad \text{(III-24)}$$

where RTIME is the mean residence time, V is the average volume over the time step, I is the average inflow over the time step, and O is the average outflow over the time step (Nix, 1985).

Because modeling the routing must be done on a smaller time step than the nutrient transformations and uptakes, average values must be taken over the calculation time step chosen for the nutrient processes. The model makes a summation of the volumes, inflows, and outflows of each cell over each time step in the routing routine. Account is kept of the number of times the model goes through the routing routine in one nutrient calculation time step. Dividing by this number gives the average volumes and flowrates needed to find the mean residence times.

Nitrogen and Phosphorus Concentration Calculations:

The concentration of nitrogen and phosphorus in each cell at a given time is calculated on a flow-weighted average. It is calculated on the same time step that the nutrient processes are evaluated on. The initial concentrations of nitrogen and phosphorus in each cell are inputs as are the inlet concentrations at each time step. The final concentration in a given cell is equal to its initial concentration minus the amounts of nutrient loss as calculated in their separate subroutines (i.e. denitrification, plant uptake, etc.). Then the initial concentration for each cell at the next time step (I+1) is calculated. This is where the flow weighting is used. A budget must be made of inflows, outflows, and storage.

The dimensions of our budget will be volume times concentration. The sum of the inflows are the average inflows (cfm) over the time step (I) multiplied by the time step length (min), multiplied by the concentrations at the time step (I) of the cells from which the flows are coming.

The outflows are the average flows out of the cell (J,K) over the time step (I), multiplied by the time step, and multiplied by the concentration of the cell (J,K) at the time step (I).

The storage term is just the volume of water in cell (J,K) at time step (I) multiplied by the concentration in the cell at time step (I). The budget is merely the inflows plus the storage term, and minus the outflows. Dividing this budget by the new volume of cell (J,K) at the new time step (I+1) gives the concentration of the cell for that time step.

Change in Storage and Elevation:

The change in storage of each cell is calculated by a budget of the inflows and outflows over the time step. The time rate of change of storage in a cell, dS/dt, is found by adding up all of the inflows to the cell and subtracting all of the outflows from the cell. Then by multiplying the rate of change of storage by the time step over which it occurs we find the volume of water added to or lost from the cell. To find the change in elevation in the cell due to this change in storage we assume the volume is evenly distributed over the cell. With this assumption we can divide by the cell surface area to find the change in elevation. Of course, the smaller the cell we are able to use, the more accurate our assumption of even distribution will be. Adding this change in height to the preceding water elevation gives the water height in the cell used to drive the flow for the next time step.

Rainfall:

Volume

The model accounts for the addition of rainfall to the pond volume on a cell-wise basis. Rainfall is assumed to be uniform over the entire pond, and it is added in one lump sum at the end of each day (every 24 hours). The monthly rainfall is also assumed to be uniformly distributed throughout the month, therefore daily rainfall within a month is the same. In calculating the daily rainfall from the given monthly rainfall, it was assumed that there were 30.4 days/mo. (365 days/12 mos.). Monthly rainfall data was taken from the monthly historical data from 11 of the SFWMD permanent monitoring stations in the area of the Upland Detention/Retention Demonstration Project (SFWMD, 1986). Values from the 11 sites are averaged to arrive at the values used in the model. Table III-6 gives the monthly average values used in centimeters, as well as the corresponding daily rainfall rate for each month in inches per day. The historical pattern also shows a distinct five month wet season and seven month dry season with monthly rainfalls of 6.65 and 2.21 inches respectively. The month in which simulation occurs is an input, and the rainfall rate for that month is used throughout the simulation.

Quality

The rainfall quality data used was gathered from three stations within the area of the Upland Detention/Retention Demonstration Project. Historical mean values for the three sites are available for $NO_x + NH_4$, total nitrogen, ortho P, and total phosphorus (SFWMD, 1986). The model uses the average values from the three sites for total nitrogen and total phosphorus to calculate the nutrient addition due to rainfall. These values are 1.54 mg/l and 0.096 mg/l for total nitrogen and total phosphorus respectively. These values will compare favorably with those found in other studies, which can be seen in Table III-7 (SFWMD, 1986). The nutrient addition is considered uniform over the pond and is added in one lump sum at the end of each day just as rainfall volume was.

Month	Total Rain (cm/mo)	Daily Rain (in/day)	Total ET (mm/mo)	Daily ET (in/day)
January	5.125	0.066	48	0.062
February	6.69	0.087	62	0.080
March	6.51	0.084	86	0.111
April	4.645	0.060	107	0.139
May	14.098	0.182	117	0.151
June	18.51	0.240	113	0.146
July	18.81	0.243	113	0.146
August	16.805	0.218	110	0.142
September	16.21	0.210	96	0.124
October	6.79	0.088	81	0.105
November	5.01	0.065	56	0.072
December	4.59	0.059	46	0.060

Table III-6 – Monthly rainfall and ET values for South Florida used in model.

Source (Location)	Total N	Total P	Date of Collection
Nicholls & Cox, 1978 (HarpLake, Ontario, Can.)	1.91	0.105	1974
Echternacht, 1975 (Fl. Peninsula)	–	0.052–0.124	Summer 1972
Zoltek, et al., 1979 (Winter Garden, Fl.)	–	0.04	5/77 – 2/78
Zoltek, et al., 1979 (Lake Apopka, Fl.)	–	0.014	3/78 – 5/79
Davis & Wisniewski, 1975 (South Fl.)	–	0.003–1.428	7/74 – 9/74
Davis, 1981 (South, Fl.)	–	0.022–0.304	Seasonal Ranges 1972 – 1973
Joyner, 1974 (Lake Okeechobee, Fl.)	0.90	0.056	1969 – 1970
Brezonik, et al., 1969 (Central Fl.)	–	0.02–0.07	2/68 – 12/68
Present Study (Kissimmee River Basin)	1.10–2.42	0.046–0.220	1974 – 1978

Table III-7 – Total nitrgen and phosphorus concentrations in rainfall from various studies.

Evapotranspiration:

The model accounts for storage volume lost by evapotranspiration (ET) in the same way it accounted for rainfall. ET is considered uniform over the pond and it is subtracted on an equal cell-wise basis at each day's end (every 24 hours). The data used is on a monthly basis, therefore it is converted to a daily rate (in./day) in the same manner in which rainfall was. The monthly values used were predicted by Penman's equation with $\alpha = 0.05$ and $k_1 = 0.7$, and using climatological data taken at the weather station at the University of Florida's Agricultural Research and Education Center in Belle Glade, Florida. This weather data was collected for the Everglades Agricultural Area for the ten year period from 1962 - 1971 (Jones et al., 1984). Again, the month of simulation given in the input set dictates the ET value used throughout the program run.

Changes in Nitrogen and Phosphorus Concentration Due to Rainfall and ET:

The model calculates the changes in nitrogen and phosphorus concentration at the end of each day (every 24 hours) when it adds rainfall and subtracts ET. It calculates the the changes on a cell-wise basis. The model calculates the total volume of water and the total masses of nitrogen and phosphorus before rainfall and ET are introduced (mass in storage). It then uses the volume of rainfall and the concentrations of nitrogen and phosphorus in the rainfall to find the masses of nitrogen and phosphorus added by the rainfall. Adding these masses to those calculated in the storage and dividing by the volume obtained after adding the rainfall and subtracting ET gives the resulting concentration in the cell.

Off-Line Detention Pond:

Design

One of the possible and probable designs available with the model is an off-line detention pond. In contrast to a flow through pond or marsh, only a portion of the storm runoff (that required by law) is routed through the pond. The remainder of the runoff by-passes the pond and is carried in the channel directly to its ultimate receiving waters. Three different views of the pond are shown in Figures III-1, 2, and 3. Storm runoff is routed toward

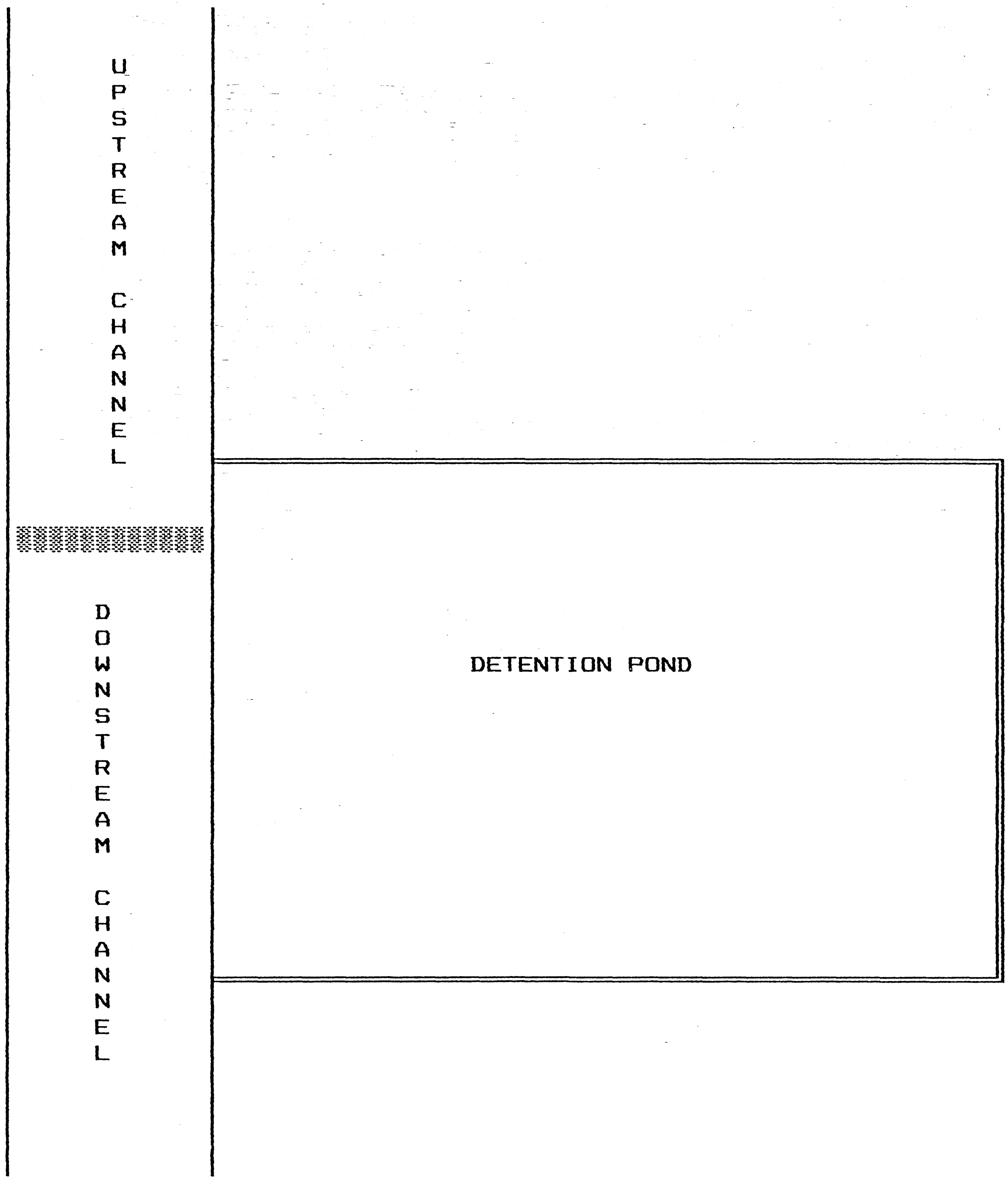

Figure III-1 – Top View of Off-line Detention Pond.

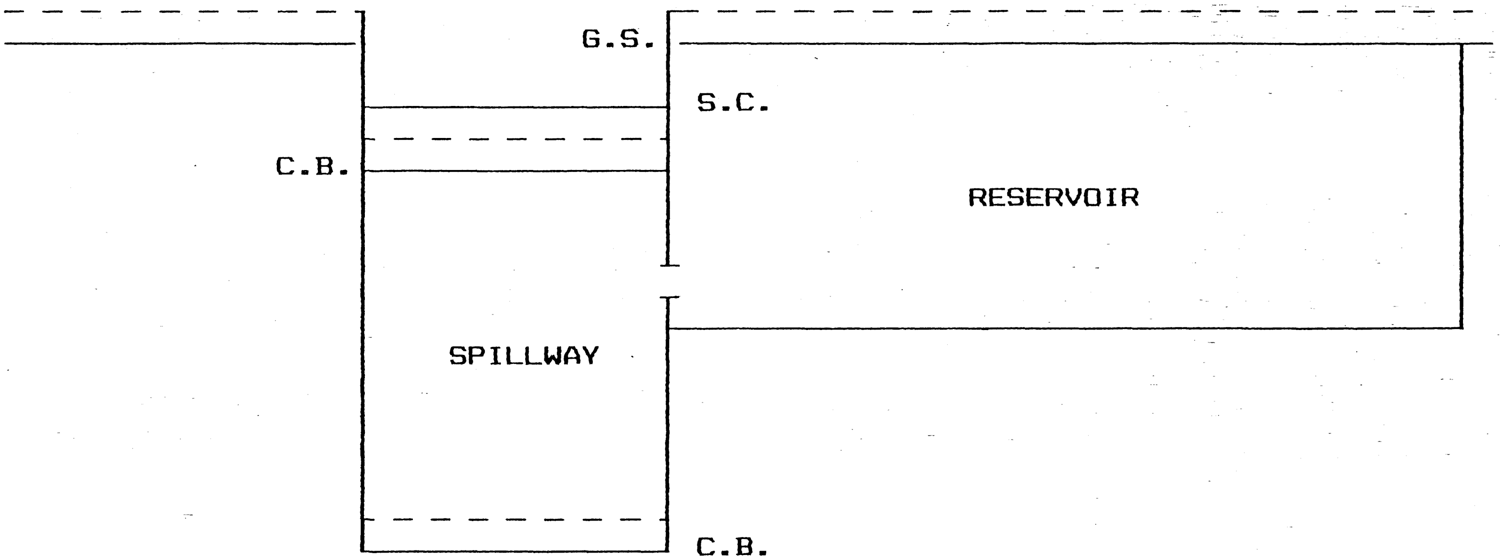

Figure III-2 – Front Cross-section of Off-line Detention Pond.

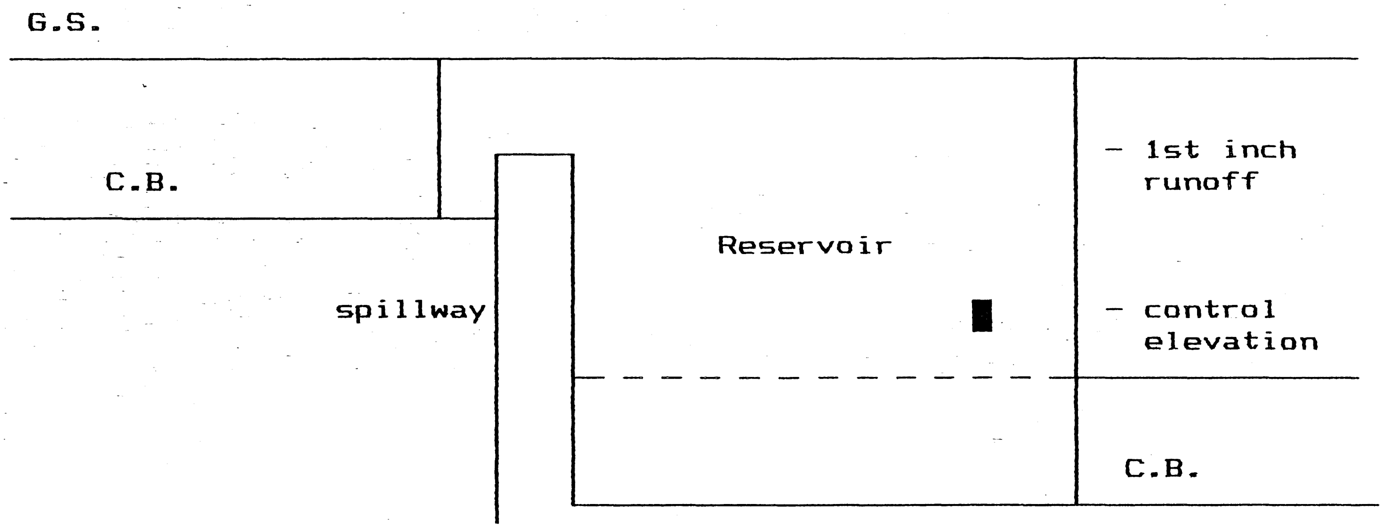

Figure III-3 - Side Cross-section of Off-line Detention Pond the pond in the upstream channel. The sides of the pond are built up as high or higher than the sides of the upstream channel. The inlet to the pond is equal in height to the channel bottom at that point, and the spillway height is slightly higher so that flow will be diverted into the pond. The pond dimensions are designed and constructed so that the volume held equivalent to the spillway crest should be equal to the volume prescribed by law for the given design storm. If this is true, then any runoff above the design volume (e.g. the first inch of runoff) will by-pass the pond, drop over the spillway, and be carried in the downstream channel to the receiving waters. The pond must also have the capability to pump or bleed-down the discharge required by legislation. Directing our attention toward the more low cost/low maintenance bleed-down method, we will put an outflow structure (weir, orifice, etc.) at a control elevation which will produce the required discharge. As the figures show, the bleed-down device should be higher than the flow in the downstream channel so that the discharge is not impeded.

Operation and Maintenance

After the original construction of such a system, operation and maintenance should be at a minimum. If gravity bleed-down is used instead of a pump, the operation and maintenance will be even less. The majority of the work will arise if a harvested system is to be used. This is highly recommended since harvesting seems to be the only real way of substantially removing nitrogen and phosphorus from the system. Also, not harvesting allows those nutrients which have been detained to be relased back into flow later. This design does have an advantage along these lines. Because the high flows of large storms by-pass the pond, losses due to flushing are greatly reduced.

The ease of harvesting should be enhanced by the regular shape of the pond. Also available is the option for a device to completely drain the pond. In certain situations this may make harvesting or maintenance easier. Under the criteria for the construction of discharge structures by the South Florida Water Management District, certain stipulations would accompany such a device. Chapter 3.2.4.1 b. states that "Discharge structures shall be fixed so that discharge cannot be made below the control elevation, except that emergency devices may be installed with secure locking devices. Either the District or an acceptable governmental agency will keep the keys for any such device." (South Florida Water Management District, 1986).

Status of Model

In closing, the model has been compiled without errors and all components run. The routing routine has been stabilized for a time step of 12-15 seconds, and the other model components have been run for short simulations with another routing procedure. The routing procedure discussed in this report must now be substituted into the model, and the model must be calibrated as well as can be with such site and storm specific phenomena. The algorithms concerning nutrient transformations, plant uptake, and plant growth have been derived from findings in the review of studies for Florida.

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